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BEHAVIOR OF THE CAST-IN-PLACE SPLICE REGIONS OF SPLICED I-GIRDER BRIDGES

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BEHAVIOR OF THE CAST-IN-PLACE SPLICE REGIONS OF SPLICED I-GIRDER BRIDGES

by

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Dedication

To my wife Gail for her continual love, support, and steadfastness

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Behavior of the Cast-in-Place Splice Regions of Spliced I-Girder Bridges

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Spliced girder technology continues to attract attention due to its versatility over traditional prestressed concrete highway bridge construction. Relatively limited data is available in the literature, however, for large-scale tests of post-tensioned I-girders, and few studies have examined the behavior of the cast-in-place (CIP) splice regions of post-tensioned spliced girder bridges. In addition to limited knowledge on CIP splice region behavior, a wide variety of splice region details (e.g., splice region length, mild reinforcement details, cross-sectional geometry, etc.) continue to be used in the field.

In response to these issues, the research program described in this dissertation was developed to (i) study the strength and serviceability behavior of the CIP splice regions of spliced I-girders, (ii) identify design and detailing practices that have been successfully implemented in CIP splice regions, and (iii) develop design recommendations based on the structural performance of spliced I-girder test specimens.

To accomplish these tasks, an industry survey was first conducted to identify the best practices that have been implemented for the splice regions of existing bridges. Splice region details were then selected to be included in large-scale post-tensioned spliced I-girder test specimens. Two tests were conducted to study splice region behavior

and evaluate the performance of the chosen details. The failure mechanisms of both test girders were characterized by a shear-compression failure of the web concrete with primary crushing occurring in the vicinity of the top post-tensioning duct. Most significantly, the girders acted essentially as monolithic members in shear at failure. Web crushing extended across much of the test span and was not localized within the splice regions.

To supplement the spliced girder tests, a shear-friction experimental program was also conducted to gain a better understanding of the interface shear behavior between precast and CIP concrete surfaces at splice regions. The findings of the shear-friction study are summarized within this dissertation.

Based on the results of the splice region research program, design recommendations were developed, including recommended CIP splice region details.

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Chapter 1. Introduction

1.1 Overview

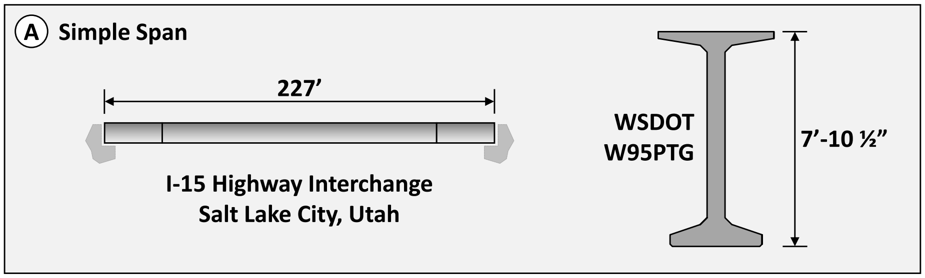
Spliced girder technology has recently attracted attention due to its versatility over traditional prestressed concrete highway bridge construction. By joining multiple precast concrete girders together using post-tensioning, spliced girder technology extends precast construction to the moderate-span market by providing a means to achieve longer span lengths than can typically be reached due to transportation restrictions. Relatively limited data is available in the literature, however, for large-scale shear tests of post-tensioned I-girders, and only a few studies have examined the behavior of the cast-in-place (CIP) splice regions of post-tensioned spliced girder bridges. A research program sponsored by the Texas Department of Transportation (TxDOT) was therefore developed to study the strength and performance of spliced girders. In the first phase of the program, the shear behavior of post-tensioned I-girders was evaluated with specific focus placed on the effect of post-tensioning ducts located within the thin girder webs. The purpose of the second phase was to study the behavior of CIP splice regions where two precast girders are joined. The details of this second phase of the spliced girder research program are the focus of this dissertation.

1.2 Overview of Spliced Girder Technology

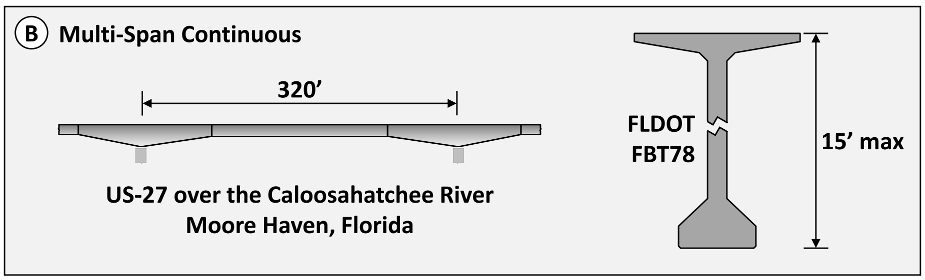
Modern spliced girder bridges typically consist of multiple precast pretensioned girders joined at short splice regions cast at the bridge site. The lengths of the individual girder segments and the resulting locations of the CIP splice regions are determined based on factors unique to each particular design scenario. The versatility of spliced girder technology coupled with the span lengths that can be achieved results in an economical alternative to other bridge types (e.g., steel plate girder, concrete segmental, and cast-in-place post-tensioned construction) in the moderate-span market.

1.2.1 Typical Applications

Spliced girder technology can be applied to both simple-span and multi-span continuous bridge construction, as illustrated in Figure 1.1. Both applications lead to significantly longer span ranges than would be possible with traditional precast girder construction. The implementation of spliced girder technology for simple-span bridges have allowed span lengths of over 200 ft to be achieved (Shutt, 1999), as indicated in Figure 1.1(a). When applied to multi-span continuous construction, spliced girder technology is allowing the precast concrete girder industry to approach its full potential with spans greater than 300 ft (Figure 1.1(b)). The splice region experimental program described in this dissertation was tailored to provide results that are applicable to various possible bridge configurations.



(a)



(b)

Figure 1.1: Examples of the application of spliced girder technology – (a) simple span; (b) multi-span continuous (from Moore, 2014)

1.2.2 Typical Construction Sequence

The application of spliced girder bridge technology introduces challenges that are not typically encountered in traditional precast girder design and construction. One of these challenges can arise from the construction sequence that must be carefully considered when designing a spliced girder bridge. Critical aspects of the construction sequence include casting schedules, use of temporary shoring or strong-backs, post-tensioning, and tendon grouting. A simplified version of a typical construction sequence for a spliced girder bridge is presented in Figure 1.2.

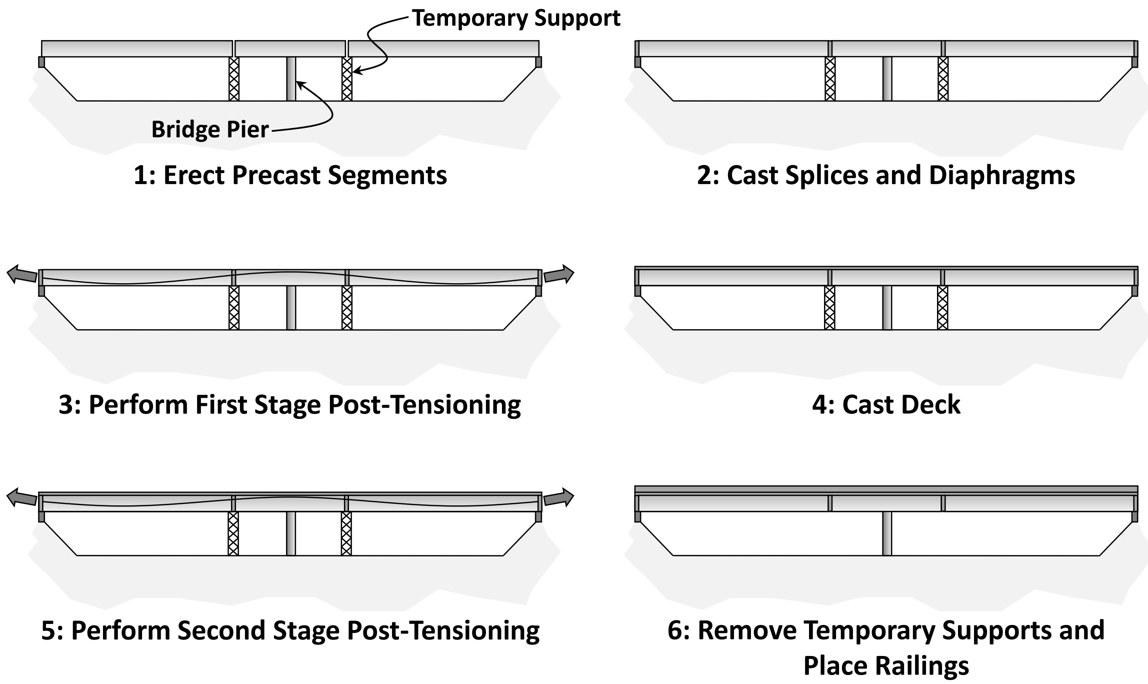


Figure 1.2: Typical construction sequence (adapted from Abdel-Karim and Tadros, 1995)

In Figure 1.2, the post-tensioning operation is conducted in two stages: one prior to and one after the placement of the deck. An alternative to this sequence is to fully stress the post-tensioning tendons before the deck is placed. This approach simplifies complete deck removal and replacement in the future (Castrodale and White, 2004). Furthermore, some construction scenarios may allow the girder splicing to be conducted on the ground prior to lifting the post-tensioned girders into their final positions.

1.3 Project Objectives and Scope

The primary objectives of the cast-in-place splice region research program were:

(i) Study the strength and serviceability behavior of the CIP splice regions of spliced I-girders.

(ii) Identify design and detailing practices that have been successfully implemented in CIP splice regions located within the span lengths of existing spliced I-girder bridges.

(iii) Develop design recommendations based on the structural performance of spliced girder test specimens that include details typical of current practice within the CIP splice regions.

To supplement the spliced girder research program, a shear-friction study was also performed and resulted in a better understanding of the interface shear behavior between precast and CIP concrete surfaces at splice regions.

Considering the limited data available in the literature for the performance of CIP splice regions, the large-scale girder tests conducted as part of the splice region research program provide significant insights into the strength and behavior of spliced girders. The test specimens were designed to exhibit a shear failure. This failure mode was selected as the most critical failure mode that is likely to be influenced by the presence of post-tensioning ducts and duct couplers. The test setup for the experimental program, however, was configured in a manner that ensured the splice regions would experience both high shear and flexural demands as the ultimate load was approached, creating a critical loading scenario.

A wide variety of splice region details (e.g., splice region length, mild reinforcement details, cross-sectional geometry, etc.) have been used in the field due to the absence of uniform design standards. To identify the best practices that have been successfully implemented for the splice regions of existing bridges, an industry survey was conducted. After analyzing the survey responses and supplementary material offered by the survey participants, splice region details to be proof tested in the laboratory were developed. Technical input offered by the TxDOT Project Monitoring Committee (PMC) and a project advisory panel were invaluable resources during the selection of the details. The project advisory panel consisted of practitioners with first-hand experience in spliced girder technology.

Two proof tests were conducted in the laboratory on large-scale spliced girder specimens to evaluate the performance of the selected splice details. The effect of the mild longitudinal interface reinforcement extending from the precast segments into the splice region was of particular interest. To study the influence of the bars on the behavior of the test girders, the interface reinforcement was varied between the two specimens. Each of the test girders was loaded monotonically until the specimen exhibited a shear-compression failure of the web concrete. Based on the results of the experimental program, design and detailing recommendations for the splice regions of spliced I-girder bridges were developed.

1.4 ORGANIZATION

A background of spliced girder design and research is provided in Chapter 2, including a literature review focusing on studies of splice region/joint performance. An overview of the first phase of the splice girder research program is also presented. In Chapter 3, the responses received from the industry survey are summarized, and a description of the survey results as they relate to the design and detailing of splice regions is given. The CIP splice region experimental program, including the specimen design and fabrication, is discussed in Chapter 4. The selection of the splice region details of the test girders is described along with the specimen instrumentation, loading configuration, and test procedure. In Chapter 5, the analysis of experimental results and observations are presented. The strength and serviceability behavior of the girders is evaluated, and the influence of longitudinal interface reinforcement is examined. In Chapter 6, the shear-friction experimental program developed to supplement the spliced girder research is introduced, and observations from eleven interface shear tests are analyzed. Design recommendations based on the results of the spliced girder research program are provided in Chapter 7, including considerations for splice region detailing and strength calculations. Lastly, the overall findings and conclusions of the research program are summarized in Chapter 8.

Chapter 2. Background of Spliced Girder Design and Research

2.1 Introduction

The background information presented in this chapter places the splice region experimental program into perspective with the current state of spliced girder design and research. Specific attention is given to the shear design and behavior of post-tensioned thin-webbed girders in Sections 2.2 and 2.3. Existing shear design provisions for post-tensioned girders are introduced. Then, the findings of the first phase of the spliced girder research program that are most relevant to the behavior of splice regions are summarized, and a proposed shear design procedure for post-tensioned girders is presented. In Section 2.4, a review of past experimental programs examining the behavior of internally prestressed splice regions/joints of spliced girders is provided.

2.2 Shear Design Provisions for Post-Tensioned Bridge Girders

2.2.1 Reduction in Effective Web Width

The presence of a post-tensioning duct in the thin-web of a bridge girder results in the girder exhibiting unique shear behaviors. The cause for the unique behaviors is the discontinuity created by the post-tensioning duct, whether empty or grouted. The effect of this discontinuity is illustrated for an I-girder in Figure 2.1. Due to the presence of a post-tensioning duct in the thin web of the girder, the compressive stresses (indicated by dashed lines in the section cut-outs) deviate from a straight-line path. The deviation of compressive stresses, either inward toward the duct or outward around the duct, results in the development of tensile stresses (represented by solid lines in the section cut-outs) through the thickness of the web. The discontinuity created by the post-tensioning duct ultimately causes a reduction in the shear strength of the girder.

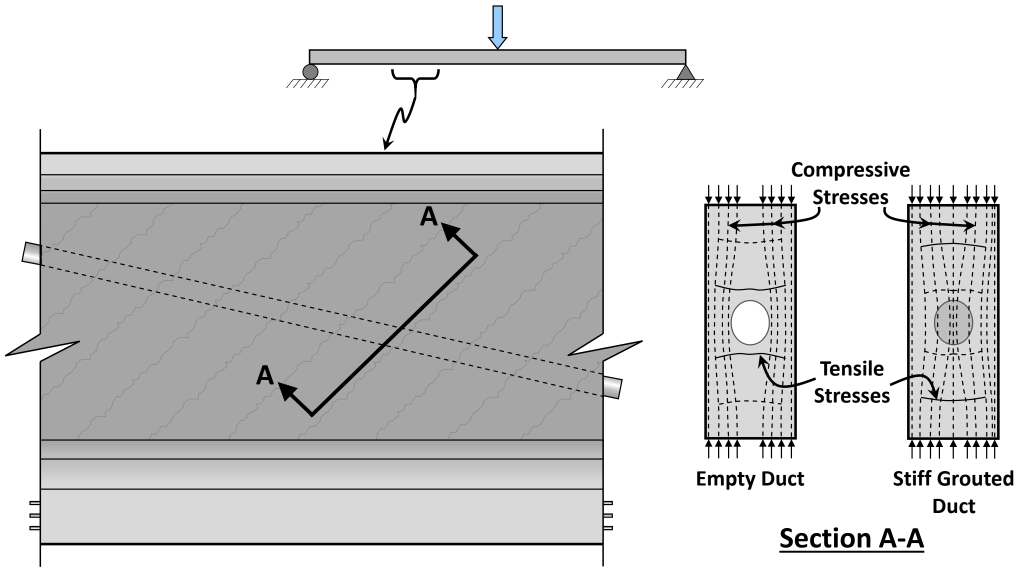


Figure 2.1: Deviation of stresses in web due to presence of post-tensioning duct (from Moore, 2014; originally adapted from Muttoni, Burdet, and Hars, 2006)

Existing shear design provisions for prestressed concrete members that account for the reduction in shear strength caused by the presence of a post-tensioning duct (e.g., *AASHTO LRFD Bridge Design Specifications* (2014), Eurocode 2) introduce the concept of an effective, or reduced, web width. For calculating the shear resistance of a member that contains a post-tensioning duct within its web, the web width is reduced based on the following expression:

$$b_v = b_w - k\text{Ø}_{duct} \tag{2.1}$$

where b_v is the effective web width to be used in shear strength calculations, b_w is the gross web width, Ø_{duct} is the diameter of the duct, and k is a reduction factor. The reduction factor, k, varies between different shear design provisions and can depend on whether the duct is grouted or ungrouted. Section 6.2.3 (6) of Eurocode 2 also differentiates between plastic and metal ducts. The reduction factor in AASHTO LRFD (2014) is provided in Article 5.8.2.9 with the following statement:

> In determining the web width at a particular level, one-half the diameters of ungrouted ducts or one-quarter the diameter of grouted ducts at that level shall be subtracted from the web width.

Referring to Equation 2.1, the value of the *k*-factor is therefore 0.5 for ungrouted ducts and 0.25 for grouted ducts. According to Article 5.8.6.1 of AASHTO LRFD (2014), these values are doubled when determining the effective web width to be used in the shear design procedure for segmental box girder bridges.

2.2.2 AASHTO LRFD (2014) General Shear Procedure

As part of the cast-in-place (CIP) splice region research, the experimental capacities of the spliced girder test specimens will be compared to their calculated shear strengths to determine if existing design provisions provide conservative strength estimates at the splice regions of the girders. Primary focus is placed on the performance of the general shear procedure of Article 5.8.3.4.2 of AASHTO LRFD (2014) and the proposed shear design provisions discussed in Section 2.3.2 below. The performance of these two procedures is emphasized based on the findings of the tests conducted as part of the first phase of the spliced girder research program and the accompanying database analysis, described in Section 2.3. Through the database analysis, Moore (2014) demonstrated that the AASHTO LRFD (2014) general shear procedure is more accurate than other existing shear design procedures when applied to post-tensioned girders (i.e., AASHTO LRFD shear design procedure for segmental box girder bridges and ACI 318-14 shear design procedures for prestressed members). The proposed provisions modify the AASHTO LRFD general shear procedure, further improving its performance.

The general shear procedure of AASHTO LRFD (2014) is based on the modified compression field theory (MCFT) defined by Vecchio and Collins (1986). As explained in the commentary to Article 5.8.3.4.2 of AASHTO LRFD (2014), although the general shear procedure was formerly iterative, the current design expressions, introduced in the

AASHTO LRFD (2008) Interim Revisions, do not require any iterations. The procedure is outlined below for the reader's convenience.

In Article 5.8.3.3 of AASHTO LRFD (2014), the shear strength contribution provided by tensile stresses in the concrete, V_c, and the contribution provided by the shear reinforcement, V_s, are calculated as follows:

$$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v \tag{2.2}$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \tag{2.3}$$

where:

A_v = Area of shear reinforcement within distance s (in.^{2})

b_v = Effective web width equal to the minimum web width within depth d_v and adjusted for the presence of post-tensioning ducts (in.)

d_e = Effective depth measured from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement (in.)

d_v = Effective shear depth equal to the distance measured perpendicular to the neutral axis between the resultants of the tensile and compressive forces due to flexure, not to be taken as less than the greater of $0.9d_e$ or $0.72h$ (in.)

f'_c = Specified compressive strength of concrete (ksi)

f_y = Specified minimum yield strength of reinforcement (ksi)

h = Overall member depth (in.)

s = Spacing of transverse reinforcement measured in the direction parallel to the longitudinal reinforcement (in.)

α = Angle of inclination of transverse reinforcement to the longitudinal axis (degrees)

β = Factor indicating ability of diagonally cracked concrete to transmit tension and shear

θ = Angle of inclination of diagonal compressive stresses (degrees)

The factors β and θ are both dependent on the net longitudinal tensile strain at the centroid of the tension reinforcement, ε_s. These factors can be determined from the expressions provided below (from Article 5.8.3.4.2 of AASHTO LRFD (2014)).

For sections containing at least the minimum amount of transverse reinforcement:

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \tag{2.4}$$

For sections not containing at least the minimum amount of transverse reinforcement:

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \frac{51}{(39 + s_{xe})} \tag{2.5}$$

For all sections:

$$\theta = 29 + 3500\varepsilon_s \tag{2.6}$$

The crack spacing parameter, s_{xe}, is calculated as:

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} \tag{2.7}$$

where:

$$12.0 \text{ in.} \leq s_{xe} \leq 80.0 \text{ in.}$$

where:

a_g = Maximum aggregate size (in.)

s_x = A parameter taken as the lesser of d_v and the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003 b_v s_x$ (in.)

For the section under consideration, the longitudinal strain, ε_s, may be calculated as follows (from Article 5.8.3.4.2 of AASHTO LRFD (2014)):

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + \left|V_u - V_p\right| - A_{ps} f_{po}\right)}{E_s A_s + E_p A_{ps}} \tag{2.8}$$

where:

A_{ps} = Area of prestressing steel on the flexural tension side of the member (in.^{2})

A_s = Area of nonprestressed steel on the flexural tension side of the member (in.^{2})

E_p = Modulus of elasticity of prestressing strands (ksi)

E_s = Modulus of elasticity of reinforcing bars (ksi)

f_{po} = A parameter taken as the modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi)

$|M_u|$ = Absolute value of the factored moment, not to be taken as less than $|V_u - V_p|d_v$ (kip-in.)

N_u = Factored axial force, taken as positive if tensile and negative if compressive (kip)

V_p = Component of the effective prestressing force in the direction of the applied shear, taken as positive if resisting the applied shear (kip)

V_u = Factored shear force (kip)

The nominal shear resistance, V_n, is then calculated as the lesser of the following expressions (from Article 5.8.3.3 of AASHTO LRFD (2014)):

$$V_n = V_c + V_s + V_p \tag{2.9}$$

$$V_n = 0.25f'_c b_v d_v + V_p \tag{2.10}$$

The limit placed on the calculated shear capacity by Equation 2.10 is intended to prevent crushing of the concrete within the girder web prior to the yielding of the shear reinforcement.

Considering Equation 2.2, the reduced web width due to the presence of a post-tensioning duct effectively reduces the V_c term, or the contribution of the concrete to the shear resistance of the member. The reduced width also affects the upper limit of V_n given in Equation 2.10. It should also be noted that the specifications impose a limit on the diameter of the duct that can placed in a girder web. Article 5.4.6.2 of AASHTO LRFD (2014) includes the following statement:

> The size of ducts shall not exceed 0.4 times the least gross concrete thickness at the duct.

Therefore, the maximum allowable duct diameter to web width ratio is 0.4. As discussed in Chapter 3, the results of the industry survey conducted for the splice region research program indicated that this limit has been routinely surpassed for the design of existing spliced girder bridges.

2.3 Summary of the Shear Behavior of Post-Tensioned I-Girders

The first phase of the spliced girder research program included large-scale tests on monolithically cast post-tensioned I-girder specimens. The primary objective of the study was to evaluate the effect of a post-tensioning duct on the shear behavior of thin-webbed girders. The influence of the duct material (steel versus plastic) on the strength and serviceability behavior of the girders was of particular interest. The experimental program included the first large-scale shear tests ever performed on internally post-tensioned bridge girders containing plastic ducts (Moore, 2014).

Relevant details of the first phase of the spliced girder research program, as reported in Moore (2014), are provided in this section. The summary of the testing program is presented, and the analysis of the Evaluation Database for Post-Tensioned Girders is discussed. Furthermore, the conclusions most relevant to the current study of CIP splice regions are provided along with the proposed shear design procedure.

2.3.1 Experimental Program

The experimental program consisted of eleven tests performed on seven large-scale prestressed concrete I-girder specimens. All seven test girders contained pretensioned strands within the bottom and top flanges. Six of the girders were post-tensioned with 12 0.6-in. diameter low-relaxation 7-wire prestressing strands that were contained in a single post-tensioning duct located at the mid-height of the web. The duct extended horizontally along the full length of the girder. A total of four tests were conducted on girder regions that contained plastic ducts, while six tests were performed on regions containing steel ducts. One girder acted as a control specimen and did not contain a post-tensioning duct.

The cross-sectional geometry of the test girders is illustrated in Figure 2.2. The section was based on the geometry of a Tx62 girder, a standard bridge girder with a height of 62 in. used in the state of Texas. Eight of the tests were conducted on specimens with web widths, b_w, of 7 in., as with typical Tx62 girders. All horizontal (i.e., transverse) dimensions were increased by 2 in. for the other three tests (b_w = 9 in.). A deck with a thickness of 8 in. was cast on each girder, resulting in a total specimen height of 70 in. The length of each girder was 50 ft. The ends of the girders were detailed with thickened end blocks to house the post-tensioning anchorage hardware and allow the post-tensioning force to be properly transferred to the concrete.

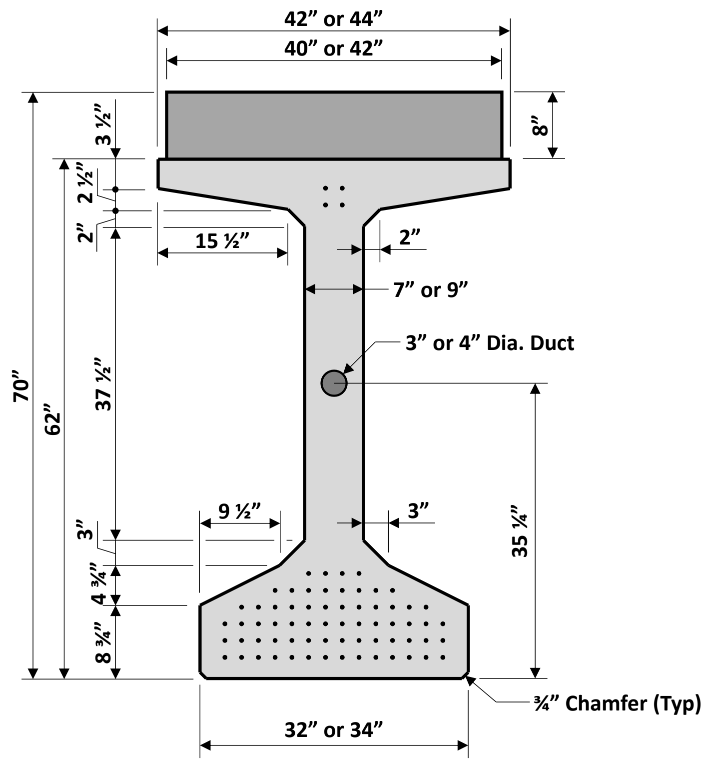


Figure 2.2: Section geometry of post-tensioned test specimens

The loading configuration used for the shear tests is shown in Figure 2.3. The configuration was slightly modified after the first test as indicated. The setup was designed to allow two tests to be conducted on each girder specimen. After the completion of the first test on the girder, the load frame and supports were moved to test the opposite end of the member. In some cases, the second test could not be performed due to the high level of damage resulting from the first test. In the end, eleven shear tests were performed on a total of seven girder specimens.

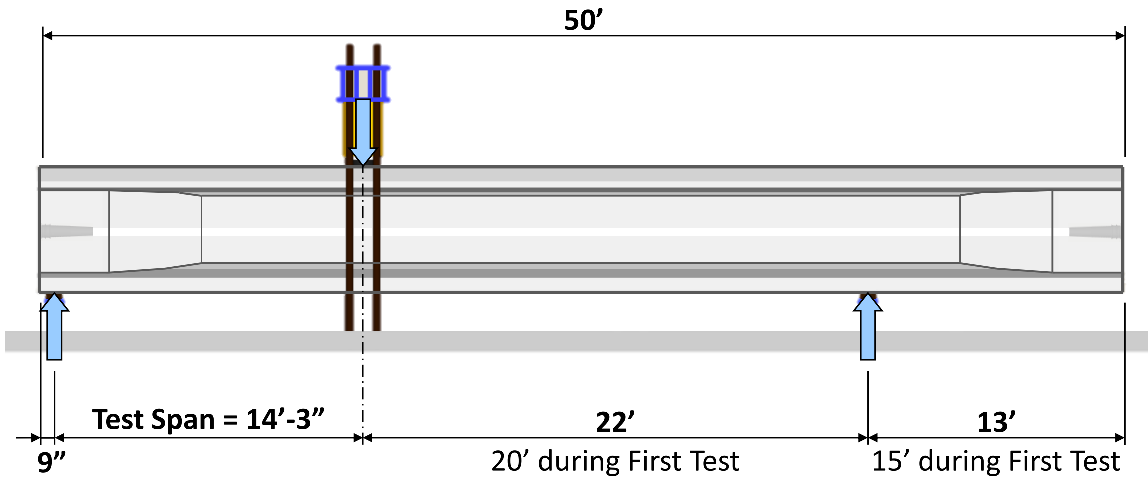


Figure 2.3: Loading configuration for the I-girder tests of the first phase of the spliced girder research program

For all eleven tests, the girders exhibited a shear failure characterized by crushing of the web concrete. The concrete crushing experienced by all of the post-tensioned girder specimens was primarily located in the vicinity of the post-tensioning duct, as indicated by one of the post-tensioned girders in Figure 2.4(a). Failure of the control specimen with no post-tensioning duct, however, was characterized by crushing within the compression field over a large portion of the girder web, as shown in Figure 2.4(b).

(a) *(b)*

Figure 2.4: I-girder specimens after failure – (a) with post-tensioning duct; (b) without post-tensioning duct (adapted from Moore, 2014)

One of the most significant findings of the experimental program was that the duct material resulted in no notable differences in the behavior of the girders. Current Eurocode 2 provisions specify a web width reduction factor, *k*, for grouted plastic ducts of 1.2. This value is 2.4 times larger than the *k*-factor specified for grouted steel ducts ($k = 0.5$). The large-scale girder tests, however, revealed that differentiating between the duct materials is unnecessary. The duct material had no notable effect on the behavior of the girders under service-level shear loads, and no influence of the material on the maximum shear stress resisted by the girders was observed. Furthermore, all post-tensioned girders experienced a shear-compression failure mechanism with primary crushing occurring in the vicinity of the duct (see Figure 2.4(a)), regardless of whether the duct was made of steel or plastic (Moore, 2014).

2.3.2 Database Analysis and Proposed Provisions

As outlined in Moore (2014), the database of post-tensioned girder specimens analyzed as part of the spliced girder research program was developed from the existing University of Texas Prestressed Concrete Shear Database (UTPCSDB) consisting of 1,696 tests. Details of the assembly of the UTPCSDB are provided in Avendaño and Bayrak (2008) and Nakamura, Avendaño, and Bayrak (2013). During a literature search conducted as part of the spliced girder research program, an additional 34 tests were identified and added to the UTPCSDB. The resulting database of 1,730 tests was then filtered to create the Post-Tensioned Girder Database consisting of 443 tests. The specimens in the Post-Tensioned Girder Database all contained post-tensioning ducts within the girder web at the shear span. Finally, additional filtering criteria were applied to the database of 443 tests to create the Evaluation Database for Post-Tensioned Girders, as illustrated in Figure 2.5. The criteria for specimens to be included in the evaluation database are outlined in the figure and were selected to include the post-tensioned specimens most representative of actual field members. Inclusion of the ten tests conducted on post-tensioned girders as part of the spliced girder research program results in an Evaluation Database for Post-Tensioned Girders, hereafter referred to as the PT Evaluation Database, consisting of 44 tests (Moore, 2014).

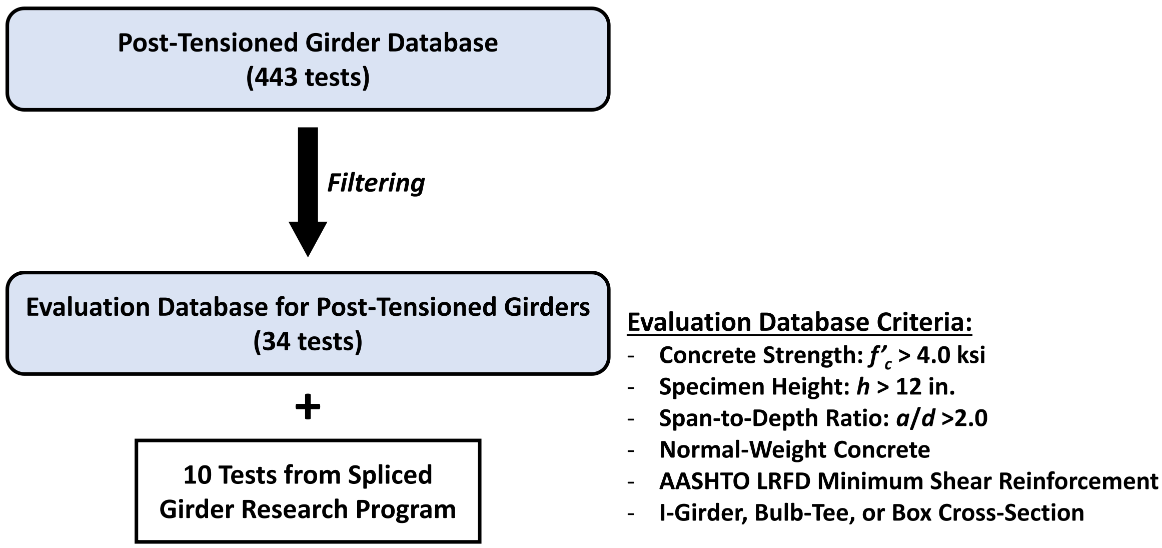


Figure 2.5: Development of the Evaluation Database for Post-Tensioned Girders (adapted from Moore, 2014)

The nominal shear strengths, V_n, of the specimens in the PT Evaluation Database were calculated using existing shear design procedures and then compared to the experimental capacities, V_{test}. As previously stated, the general shear procedure of AASHTO LRFD (2014) is more accurate than the AASHTO LRFD shear design procedure for segmental box girder bridges (Article 5.8.6.5 of AASHTO LRFD (2014)) and shear design provisions for prestressed members in ACI 318-14 (Sections 22.5.8.2 and 22.5.8.3 of ACI 318-14). Despite its relative accuracy, however, Moore (2014) reported that the AASHTO LRFD general shear procedure provided unconservative strength estimates (i.e., $V_{test}/V_n < 1.0$) for 13.6 percent of the 44 tests in the PT Evaluation Database. Considering the 164 tests within the Evaluation Database for *Pretensioned* Girders developed from the UTPCSDB (excludes all post-tensioned specimens), only one test was unconservatively estimated by the AASHTO LRFD general shear procedure (Moore, 2014). Any proposed modifications to the AASHTO LRFD specifications should therefore aim at increasing the conservatism of the general shear procedure for post-tensioned girders.

Considering the failure mechanism characterized by concrete crushing in the vicinity of the post-tensioning duct, the ability of the concrete to transmit diagonal *compression* through the web is reduced due to the presence of the duct. The concept of a reduced web width within the current AASHTO LRFD (2014) shear design provisions effectively reduces the V_c term within the expression for the nominal shear resistance, V_n (Equation 2.9). The V_c term, however, represents the shear force resisted by *tensile* stresses in cracked concrete (Collins et al., 1996). Therefore, for the observed failure mechanism of the post-tensioned girders, reducing the web width to account for the effect of a post-tensioning duct is an incorrect approach, as explained by Kuchma (2011).

To help illustrate the reduced shear strength resulting from the presence of a post-tensioning duct within the web of a girder, the contribution of the V_s term to the nominal shear resistance is represented in Figure 2.6 by a simple two-panel truss analogy (Moore, 2014). The shear strength contribution of the tensile stresses in the transverse reinforcement, represented by the V_s term, is limited by the ability of the concrete to transmit diagonal compressive stresses through the web. Since a post-tensioning duct reduces the ability of the concrete to transmit diagonal compression, a reduction of the V_s term, not the V_c term, is appropriate to account for the presence of the duct (Kuchma, 2011). Based on this logic, design provisions were proposed by Moore (2014) that modify the AASHTO LRFD (2014) general shear procedure to include a reduction factor that is applied to the V_s term.

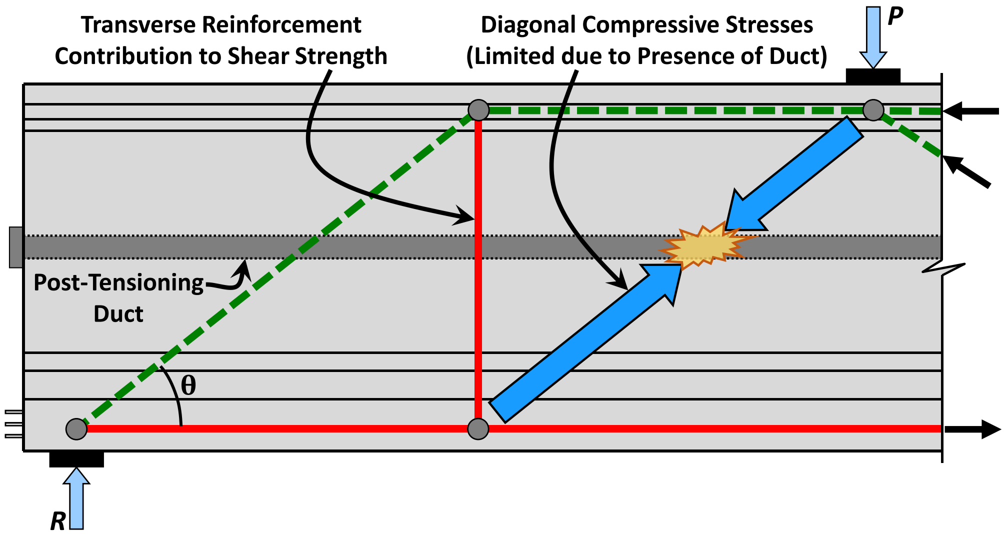


Figure 2.6: Truss analogy illustrating reduction in shear strength due to presence of post-tensioning duct (adapted from Moore, 2014)

The proposed reduction factor was calibrated using the PT Evaluation Database while considering the current AASHTO LRFD (2014) limit of 0.4 placed on the duct diameter to web width ratio. As previously mentioned, this limit is often surpassed in practice. Furthermore, all of the specimens in the PT Evaluation Database with strengths that were unconservatively estimated by the AASHTO LRFD general shear procedure contained ducts that exceeded this limit (Moore, 2014). A reduction factor was therefore recommended by Moore (2014) that decreases rapidly as the duct diameter to web width ratio approaches large values. The proposed shear strength reduction factor, λ_{duct}, is defined by the following quadratic expression and represented graphically in Figure 2.7:

$$\boldsymbol{\lambda_{duct} = 1 - \delta\left(\frac{\varnothing_{duct}}{b_w}\right)^2} \tag{2.11}$$

where $\varnothing_{duct}$ is the diameter of the post-tensioning duct and b_w is the gross web width. The duct diameter correction factor, δ, was calibrated using the PT Evaluation Database and is equal to 2.0 for grouted plastic and steel ducts. With the inclusion of this reduction factor in the AASHTO LRFD (2014) general shear procedure, the current limit on the duct diameter to web width ratio can be increased.

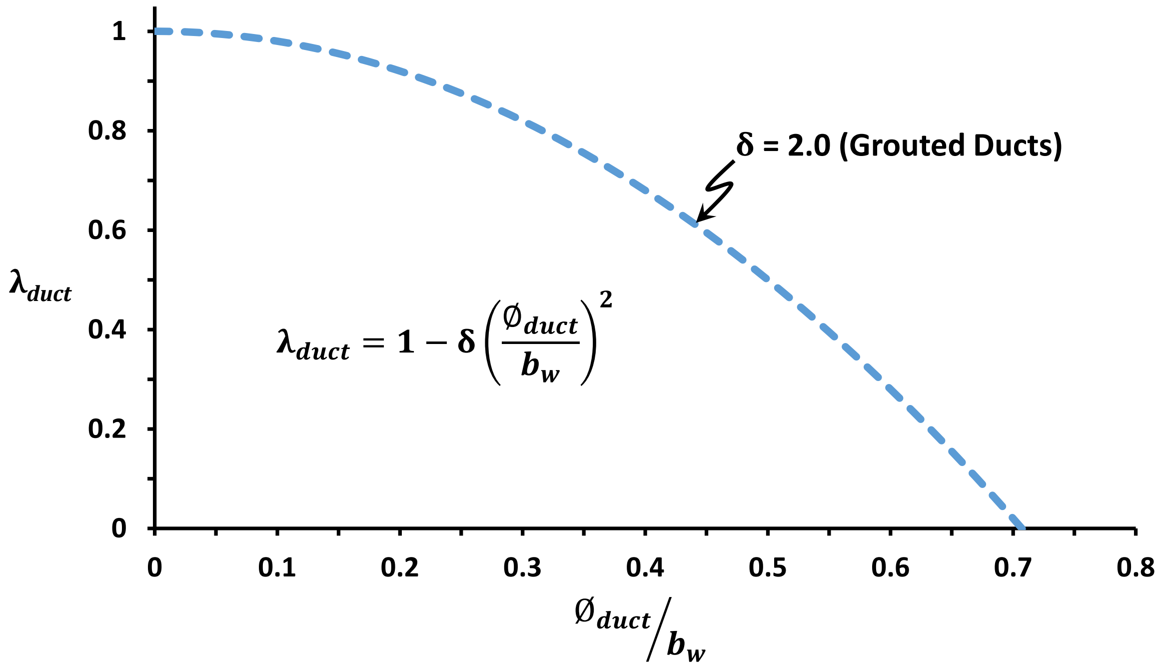


Figure 2.7: Strength reduction factor, λ_{duct} (from Moore, 2014)

Applying λ_{duct} to the V_s term allows the strength reduction currently placed on the V_c term to be removed. The effective web width, b_v, is replaced with the gross web width, b_w in both the expression for V_c (Equation 2.2) and the upper limit imposed on the nominal shear resistance, V_n (Equation 2.10). The nominal shear resistance is therefore calculated as presented below. The proposed shear design procedure is outlined in Appendix E for the reader's convenience.

The nominal shear resistance, V_n, is calculated as the lesser of the following expressions:

$$V_n = V_c + V_s + V_p \tag{2.12}$$

$$V_n = 0.25 f'_c \boldsymbol{b_w} d_v + V_p \tag{2.13}$$

where:

$$V_c = 0.0316 \beta \sqrt{f'_c}\, \boldsymbol{b_w} d_v \tag{2.14}$$

and

$$V_s = \frac{\boldsymbol{\lambda_{duct}} A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \tag{2.15}$$

A summary of the performances of both the AASHTO LRFD (2014) general shear procedure and the proposed provisions is provided in Table 2.1 in terms of the shear strength ratio, V_{test}/V_n, for tests in the PT Evaluation Database. Three tests within the database were conducted on specimens with ungrouted ducts and are therefore not included within the table. With the application of the proposed procedure, the number of tests corresponding to unconservative strength estimates is reduced from six tests to only one test with a shear strength ratio of 0.97. The proposed provisions also result in slightly less scatter in the data as indicated by the values of the standard deviation and coefficient of variation (COV).

Table 2.1: Performance of Sectional Shear Design Provisions – Shear Strength Ratio, V_{test}/V_n, Statistics (adapted from Moore, 2014)

	Design Provisions	
***n* = 41 tests**	**AASHTO LRFD (2014) General Procedure**	**Proposed Procedure**
Maximum	2.13	2.15
Minimum	0.92	0.97
Mean	1.34	1.46
Number Unconservative*	6 tests	1 test
Percent Unconservative*	14.6%	2.4%
Standard Deviation	0.36	0.34
COV**	0.27	0.23

*Unconservative = $V_{test}/V_n < 1.0$
**COV = Coefficient of Variation = Standard Deviation/Mean

In the proposed provisions introduced by Moore (2014), the upper limit imposed on the nominal shear resistance, V_n, is expressed by Equation 2.13. It should be noted that relatively few specimens within the PT Evaluation Database exceeded this limit or the current limit in the AASHTO LRFD (2014) general shear procedure. Considering the possible design implications of the proposed upper limit placed on V_n, further evaluation and testing may be needed to assess the limit and determine if additional refinement is needed.

2.4 Past Spliced Girder Experimental Studies

Considering the documents available within the literature related to spliced girder technology, primary interest was placed on experimental studies conducted to examine the behavior of the splice regions/joints of spliced girder bridges. A limited number of studies that included tests of spliced precast girders with internally prestressed splice regions/joints have been identified. Relevant research findings of such studies are described in this section. (The studies have been ordered chronologically.) It should be noted that no tests studying the shear failure mechanism of spliced girders containing post-tensioned in-span CIP splice regions were identified, emphasizing the value of the tests conducted as part of the splice region research program described in the following chapters.

2.4.1 Garcia (1993)

Garcia (1993) described the implementation of a post-tensioned bulb-tee girder system in Florida. Tests conducted on a full-scale prototype girder line are highlighted. The tests were performed prior to the full production of the bulb-tee girder system for the Eau Gallie Bridge near Melbourne, Florida. The specimen consisted of two 145-ft long girders spliced together at an intermediate support (see Figure 2.8) by continuous post-tensioning. Failure of the specimen was governed by flexure at the intermediate support. The specimen exhibited highly ductile behavior of the post-tensioned bulb-tee girder system and a capacity significantly greater than the design moments.

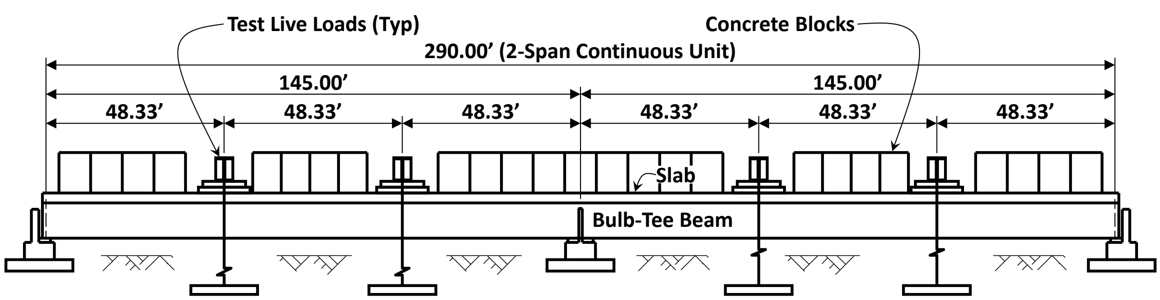


Figure 2.8: Elevation view of prototype girder line (adapted from Garcia, 1993)

2.4.2 Tadros et al. (1993)

Tadros et al. (1993) introduced a splicing method that relies on continuity provided by coupled pretensioned strands extending for the ends of precast girder segments. The proposed method is illustrated in Figure 2.9. The precast pretensioned segments are first fabricated. The ends of the segments are coped to accommodate strand splicing. The strand extensions from the top flanges of adjacent girders are then spliced using mechanical connectors. Next, hydraulic rams are installed and used to push outward on the girder segments, imposing tensile forces on the coupled strand extensions. Following the jacking procedure, a closure pour is performed within the splice region. After the concrete reaches its required strength, the jacking force is released, introducing compression to the splice concrete. The proposed method was intended to be implemented at interior supports to provide continuity between adjacent bridge spans without the use of continuous post-tensioning.

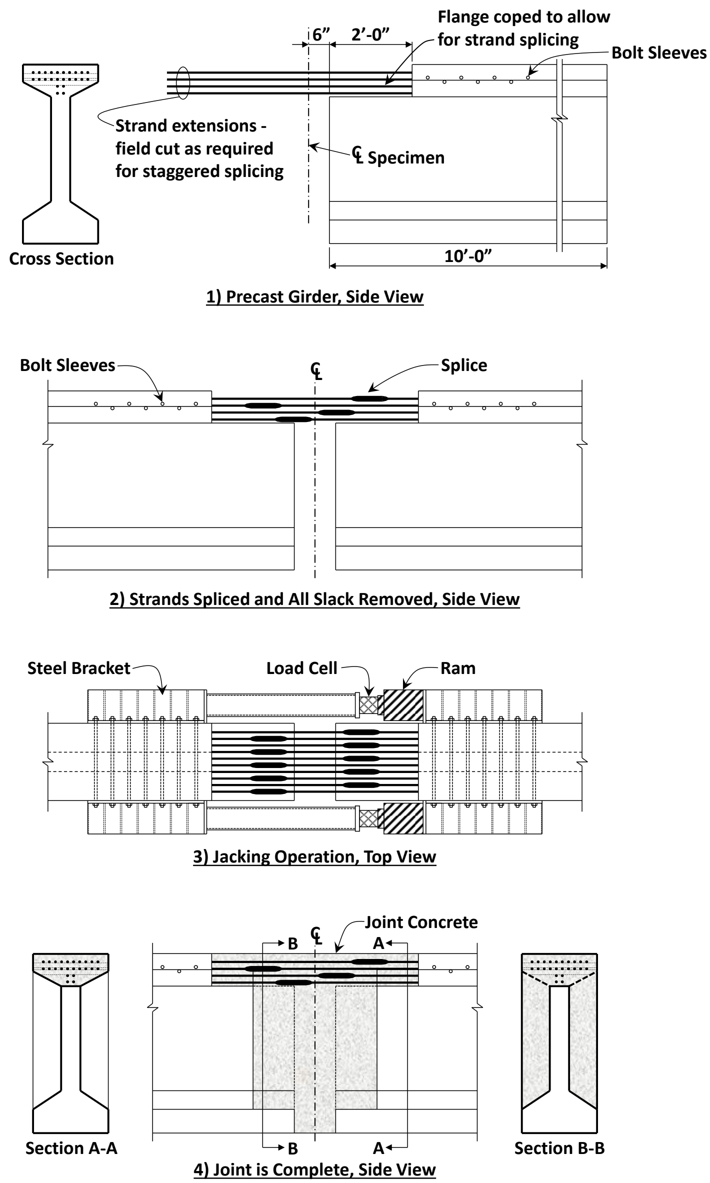


Figure 2.9: Splicing operation of test specimen (adapted from Tadros et al., 1993)

A load test was conducted on a specimen consisting of two 10-ft long precast I-girders spliced together using the proposed detail. The specimen was simply supported, and load was applied at the location of the splice region (i.e., at the midspan). The specimen exhibited a shear failure, as shown in Figure 2.10. The failure occurred at an applied load of 390 kips, while the estimated shear capacity corresponded to a load of 360 kips. The authors reported that the single flexural crack that formed during testing completely closed upon release of the applied load, demonstrating the prestressing force that was effectively applied to the joint.

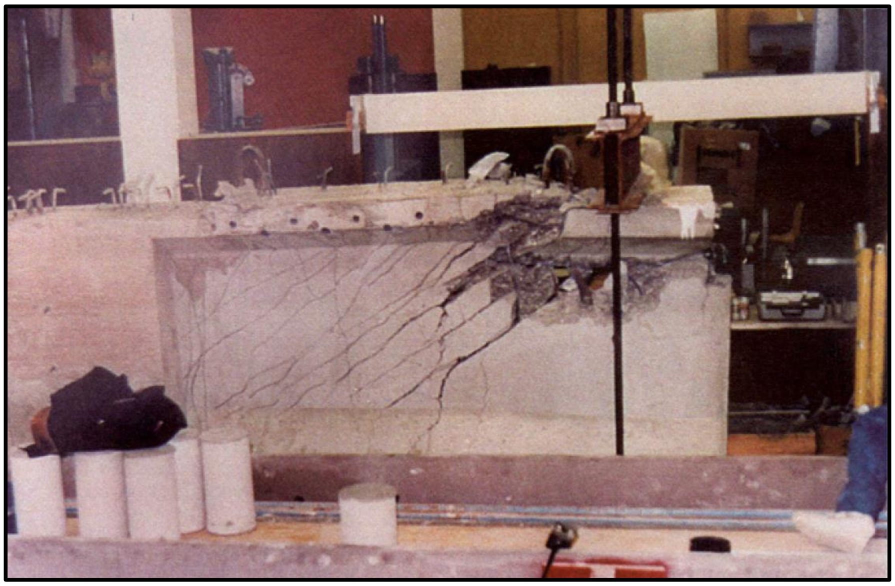


Figure 2.10: Test specimen after failure (from Tadros et al., 1993)

2.4.3 Holombo, Priestley, and Seible (2000)

Holombo, Priestley, and Seible (2000) studied the performance of spliced girders with superstructure-column continuity. Primary focus was placed on the design and behavior of the connection between the girders and the bent cap under seismic loads. Two model bridge structures were tested to evaluate the structural performance. The test setup for the 40-percent scale specimens is shown in Figure 2.11. One of the models incorporated precast post-tensioned bulb-tee girders that passed continuously through the bent cap. The other model used precast bathtub girders that were discontinuous at the bent cap. The bathtub girders were therefore spliced with continuous post-tensioning at the supporting bent. The horizontal actuators illustrated in Figure 2.11 were used to subject both specimens to forces and displacements simulating seismic activity.

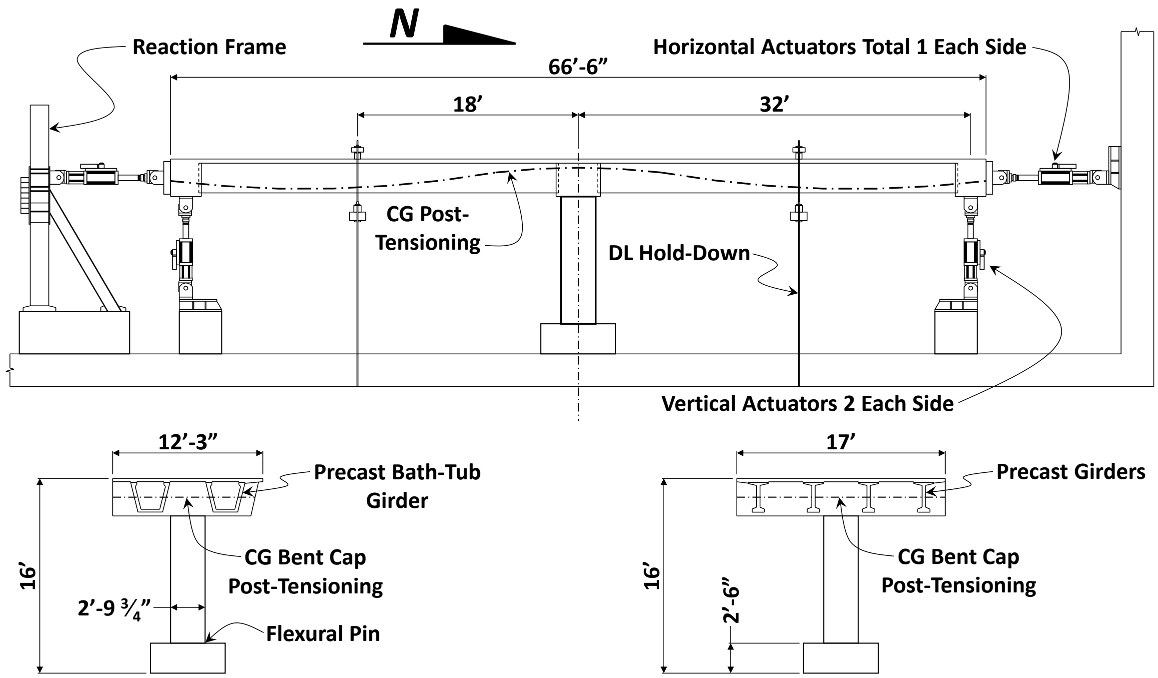


Figure 2.11: Test setup for model bridge specimens (adapted from Holombo, Priestley, and Seible, 2000)

Both model bridge specimens exhibited satisfactory ductile behavior under forces and displacements exceeding seismic design requirements, demonstrating the potential of the superstructure-column connection. The bent cap and superstructure displayed an essentially elastic response throughout the tests with the development of only minor cracks. An additional superstructure capacity test was later performed on each specimen using a different test setup designed to apply vertical excitation (Holombo, Priestley, and Seible, 1997). The superstructures also exhibited ductile behavior under these simulated seismic demands.

2.4.4 Kim, Chung, and Kim (2008)

Kim, Chung, and Kim (2008) performed load tests on two precast post-tensioned box girder specimens, one cast monolithically and the other consisting of three spliced girder segments. The precast segments of the spliced girder specimen were match-cast. Several mechanical shear connectors, shown in Figure 2.12, were installed in the ends of adjacent segments to aid in shear transfer at the match-cast joints. Epoxy resin was also applied to the joint surfaces. Continuity between the precast segments was provided by continuous post-tensioning. During the load tests, both the monolithic and spliced girders were simply supported with a span of 65 ft. The applied loads were centered on the span to evaluate and compare the flexural performance of the two specimens. The maximum load applied to the specimens was governed by the capabilities of the laboratory equipment, and the specimens did not reach their ultimate capacities.

Figure 2.12: Mechanical shear connector used at match-cast joints (from Kim, Chung, and Kim, 2008)

During the tests, both specimens exhibited similar behavior under service loads and within the elastic range. The spliced girder experienced a reduced flexural stiffness compared to the monolithic girder at higher loads. Failure of the joints of the spliced girder was not observed, and the measured relative vertical displacements between adjacent precast segments were negligible up to the maximum applied load. Chung and Kim (2011) evaluated the dynamic properties of the same two specimens both before and after the flexural load tests were performed. The findings revealed that the spliced and monolithic girders exhibited similar dynamic characteristics in both cases.

2.4.5 Han et al. (2010, 2014)

Han et al. (2010, 2014) introduced a new type of spliced post-tensioned bridge girder characterized by equally spaced holes in the web along the length of the member. In addition to reducing the self-weight of the girder, the holes accommodate post-tensioning anchorages (Figure 2.13(a)). Post-tensioning can therefore be applied at the ends of the girder as well as at the holes. With the anchorages distributed along the length of the member, less prestressing force is introduced at the girder ends compared to conventional post-tensioned girders.

(a) *(b)*

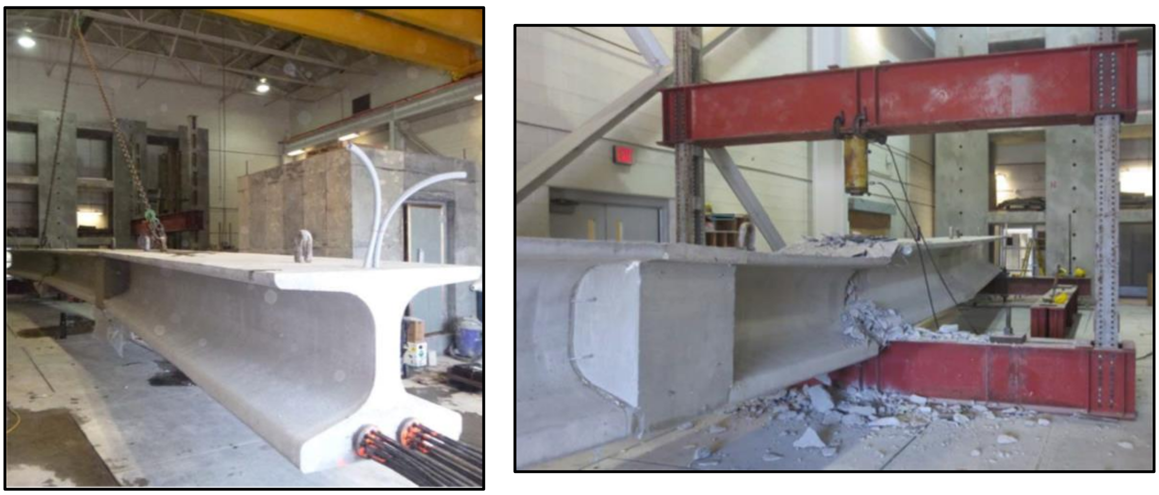
Figure 2.13: Post-tensioned girder with holes in the web – (a) post-tensioning anchorages at holes; (b) test setup (from Han et al., 2010)

The researchers tested two girder specimens, each with a total length of 50 m (164.0 ft) and a height of 2 m (6.6 ft). One of the girders was cast monolithically while the other consisted of five 10-m (32.8-ft) long segments spliced together at epoxied joints. Each joint was detailed with multiple shear keys, and no mild reinforcing bars crossed the joints. The post-tensioning tendons were concentrated in the bottom flange of the girders and were stressed in two stages (before and after placement of a deck). For each test, the girders were simply supported with two concentrated loads centered on the span (Figure 2.13(b)).

Han et al. (2014) reports that the load-deflection behavior of the two girders was similar through the full range of applied loads. When the design live load was applied to the spliced girder specimen, the separation of the girder segments measured at the joints was very small (0.06 mm at the joint closest to the applied load). One of the most significant observations related to the joint behavior was that the diagonal shear cracks that formed in the web extended across the joints, indicating structural continuity. The spliced girder is reported to have exhibited satisfactory behavior up to the maximum load of 2,500 kN (562.0 kips) applied during the test.

2.4.6 Alawneh (2013)

Alawneh (2013) proposed a system to be used as an alternative to traditional curved bridge superstructures. The system consists of relatively short straight-line precast concrete girder segments that are post-tensioned together at kinked joints (i.e., splice regions) to mimic the shape of truly curved girders. As part of the research program, two specimens were fabricated and tested. Each specimen was constructed from three precast girder segments that were joined together at splice regions. The precast segments were not pretensioned. Instead, the primary longitudinal reinforcement consisted of two post-tensioning tendons that were contained within the bottom flanges of the specimens. The straight-line segments were oriented in a manner mimicking a curve with a radius of 200 ft. One of the specimens was comprised of three tub-girder segments, while the other specimen consisted of I-girder segments (Figure 2.14(a)). The two splice regions within the I-girder specimen were of different shapes. One matched the cross-section of the adjacent segments, while the other extended out to at least the bottom flange of the girders.



(a) *(b)*

Figure 2.14: I-girder test specimen with two splice regions – (a) during lifting; (b) after failure (from Alawneh, 2013)

Each of the post-tensioned girder specimens had a test span of 60 ft with a single point load applied at midspan. Only the I-girder specimen was loaded to failure. The flexural failure under the load point is presented in Figure 2.14(b). Finite element analyses were also performed in addition to the load tests. From the results of the research program, the author states that shear-friction principles can be used to design the splice regions if stresses from both shear and torsional effects are considered.

2.4.7 Brenkus and Hamilton (2013)

Brenkus and Hamilton (2013) evaluated the performance of a girder splicing procedure with similar concepts to the method introduced by Tadros et al. (1993). A total of nine specimens with an AASHTO Type II cross-section were tested. Three of the specimens were cast monolithically, while each of the remaining six specimens consisted of two precast segments joined at a cast-in-place splice region (see Figure 2.15). Continuity was provided between the precast girder segments through the coupling of prestressing strands extending from the bottom flanges. Tensile forces were applied to the strands by using hydraulic cylinders placed on the sides of the girder webs. For each pair of precast segments, one contained five fully bonded pretensioned strands. The other segment contained only one bonded pretensioned strand. Another five strands, unbonded along the segment length, were coupled with the strands extending from the first segment during the splicing operation. The full-scale application of the splicing procedure was intended for a simply-supported span length greater than 200 ft.

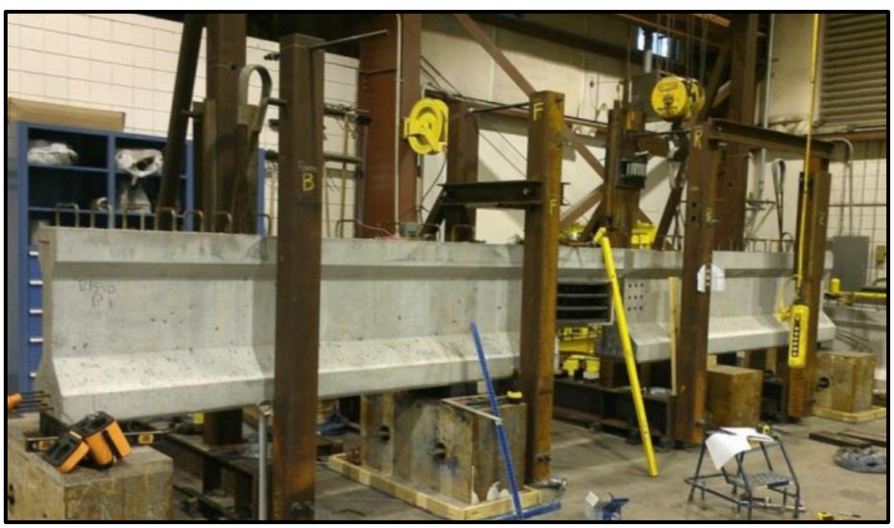


Figure 2.15: Girder segments in assembly frame (from Brenkus and Hamilton, 2013)

During the testing program, the flexural, shear, and fatigue performance of the spliced and monolithic (i.e., control) girder specimens was studied. The flexural tests demonstrated that the failure loads of the spliced specimens exceeded the calculated capacity according to AASHTO LRFD (2007) by 15 percent if bonded strands were assumed and by 24 percent assuming unbonded strands. Although a different test setup was configured to study the shear behavior of the girders, only one of the specimens exhibited a shear failure. The failure was characterized by slip at the vertical interface between the splice region and the segment containing unbonded strands. The ratio of the applied ultimate shear force to the shear capacity predicted by the AASHTO LRFD (2007) general shear procedure (Article 5.8.3.4.2) was reported to be 1.03.

The researchers also observed that the application of epoxy on the ends of the precast girder segments prior to the closure pour resulted in a noticeable improvement to the bond between the cast-in-place concrete and the precast segments. Furthermore, the authors noted the importance of adequate vibration to properly consolidate the concrete within the congested splice regions, even with the use of a self-consolidating mixture.

2.5 SUMMARY

Current shear design provisions that account for the reduction in shear strength due to the presence of a post-tensioning duct reduce the width of the web considered to be effective in resisting shear stresses. During the first phase of the spliced girder research program, existing shear design procedures were examined, and updated design provisions that are based on observations from experimental tests were proposed. The recommended provisions include a reduction factor that is applied to the shear strength contribution provided by the transverse reinforcement, V_s, instead of reducing the effective web width. The eleven tests conducted as part of the research resulted in significant insights into the influence of a post-tensioning duct on the shear strength of thin-webbed prestressed girders.

Although limited research focusing on splice region behavior has been conducted, experimental programs that included tests on spliced girders with internally prestressed splice regions/joints were identified, and relevant findings from such studies were presented in the previous sections. Despite the value of past spliced girder research, many questions concerning the behavior of CIP splice regions remain unanswered. Additional research examining the behavior of spliced girder bridges and CIP splice regions can lead to refinements of current design procedures and detailing practices and therefore provide the tools necessary for spliced girder technology to better reach its full potential.

In the following chapter, an overview of the industry survey conducted as part of the spliced girder research is presented. The splice region experimental program is then introduced and detailed in Chapter 4.

Chapter 3. Industry Survey

3.1 Overview and Objective

An industry survey was conducted to identify design and detailing practices that have been successfully implemented in cast-in-place (CIP) splice regions located within the span lengths of existing spliced I-girder bridges. The survey participants were given the opportunity to provide information regarding standard detailing practices as well as their overall experiences with the implementation of spliced girder technology. The information from the survey responses were used to develop the splice region details for the large-scale spliced girder testing program, as described in Chapter 4.

The survey was distributed in the spring of 2013 to every state Department of Transportation (DOT) in the U.S. outside of Texas. Responses from 24 states and the District of Columbia were received. Answers to the narrowly focused questions on the survey provided valuable information concerning typical details of CIP splice regions. Among the state DOTs that have experience with the design and construction of spliced I-girder bridges, the survey revealed that the details of splice regions vary significantly. From the in-depth responses and supplementary material provided by the survey participants, however, successful practices for the design and construction of splice regions could be identified. The responses from the survey participants are provided in Appendix A.

3.2 Experience with Spliced Girder Technology

The first section of the industry survey was used to identify each state DOT's experience in the design and construction of spliced girder bridges. Out of the 25 responses, 12 state DOTs have had experience with the technology. The experience of these state DOTs in the design and/or construction of spliced U-girder or box girder bridges and of spliced I-girder bridges is summarized in Figure 3.1. Although the Minnesota Department of Transportation (MnDOT) has had experience with spliced I-girder bridges, the experience was not current. Therefore, MnDOT did not provide answers to the survey questions that followed. According to the survey responses, the states where the most spliced I-girder bridges had been constructed were California, Florida, and Washington, each with over 20 bridges. For the remainder of the survey, the participants with experience in spliced I-girder bridge construction were asked to focus on the design and construction details for this specific bridge type.

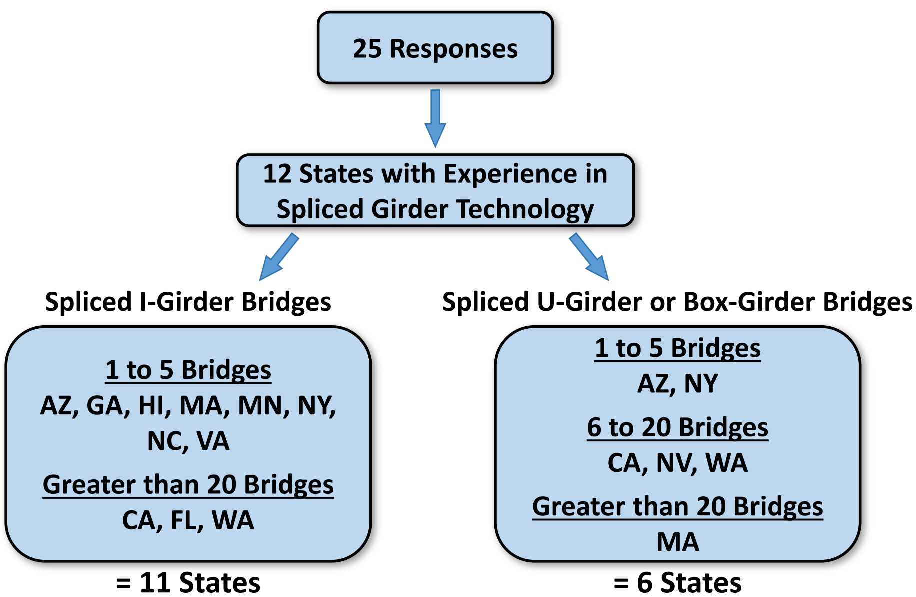


Figure 3.1: State DOT experience with spliced girder technology

3.3 Design and Construction Practices of Spliced I-Girder Bridges

The main body of the industry survey focused on the design and construction practices of spliced I-girder bridges, with specific attention given to CIP splice regions. For each detail examined through the survey, the key observations gathered from the responses are provided in the following sections. Responses from the 10 state DOTs that have had recent experience with spliced I-girder bridge design and/or construction receive primary focus.

3.3.1 Duct Material

The post-tensioning tendons of spliced girder bridges are generally contained within either galvanized steel ducts or plastic ducts. The survey participants were asked to provide the percentage of spliced girder bridge projects in their state/district for which each type of duct material had been specified. If one material was preferred over the other, the participants were also asked to explain why. They were then given the opportunity to describe any problems that may have been encountered due to the duct material that was chosen for a particular project. Considering the usage of each duct material along with the accompanying comments, it can be inferred that 7 state DOTs prefer steel post-tensioning ducts, while only 3 DOTs prefer plastic ducts. The reasons given by the survey participants for preferring a particular duct material are summarize in Table 3.1.

Table 3.1: Reasons for Preferring Steel or Plastic Ducts

Steel Ducts	Plastic Ducts
• Require less support to prevent misalignment and displacement during casting (reference was made to Castrodale and White, 2004) • Offer ease of placement • Fit better within the web width because of ducts' exterior dimensions	• Less prone to corrosion • Provide better durability • Can be sealed better • Have a smaller chance of being damaged during construction

3.3.2 Consideration of a Reduction in Shear Strength due to Presence of Ducts

The following question on the survey inquired whether the states/districts consider a reduction in shear strength due to the presence of post-tensioning ducts within girder webs. The responses revealed that only 4 of the state DOTs with experience in spliced I-girder technology reduce the shear strength to account for the effect of the ducts. The other 6 DOTs do not consider the strength reduction.

3.3.3 Grouted versus Ungrouted Ducts

The survey participants were asked if their state/district has ever used ungrouted ducts in spliced I-girder construction. The responses indicated that none of the state DOTs have used ungrouted ducts within spliced I-girders.

3.3.4 Shear Interface Detail

Drawings of existing spliced I-girder bridges reveal that various concrete surface details have been specified at the interface between the precast concrete segments and the CIP splice regions. The primary purpose of the surface details are to aid in transferring shear between the precast concrete and the splice region concrete. The most common shear interface details identified in the drawings of existing bridges are presented in Figure 3.2.

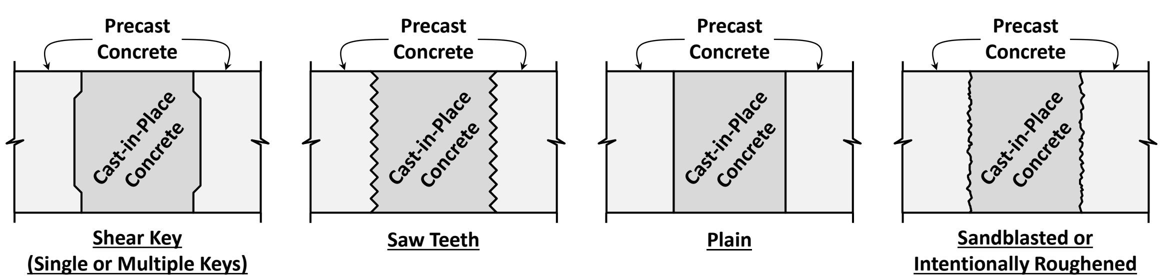


Figure 3.2: Common shear interface details

The survey participants were asked what shear interface details have been used for spliced I-girder bridges in their state/district and for what percent of projects each type of detail has been specified. The responses to these questions are summarized in Figure 3.3. For this figure, the number of spliced I-girder bridges in each state was estimated using the survey responses along with the listing of spliced girder bridge projects provided in NCHRP Report 517 (Castrodale and White, 2004). The percent of projects with each interface type as indicated by the survey responses was then used to calculate the number of bridges that are represented in Figure 3.3. As shown in the figure, shear keys are the most common shear interface detail, used in over half of the total estimated number of spliced I-girder bridges.

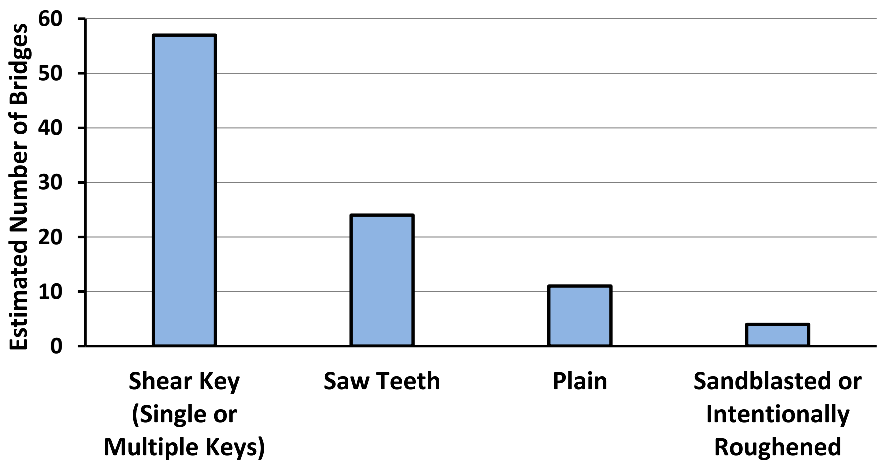


Figure 3.3: Use of the various shear interface details

The survey also provided the opportunity for the participants to explain the factors that affect the type of shear interface detail that is chosen. Only two of the responses referred to structural behavior or constructability related to the interface detail. The New York State DOT stated that that they "believe a shear key provided the best shear transfer mechanism." The North Carolina DOT explained that shear keys are a "[s]imple detail that is easy to fabricate and control during fabrication."

3.3.5 Longitudinal Interface Reinforcement

The detailing of the mild longitudinal interface reinforcement that extends from the precast concrete segments into the CIP splice regions vary significantly among existing spliced I-girder bridges. In general, this longitudinal reinforcement is provided to satisfy stress limits for the splice region at the service limit state as well as to meet shear strength requirements at the splice (Castrodale and White, 2004). Possible bar details for the interface reinforcement are presented in Figure 3.4.

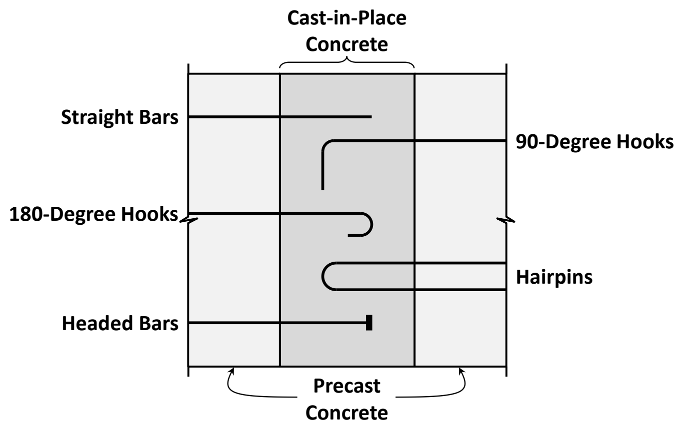


Figure 3.4: Possible bar details for longitudinal interface reinforcement

To determine the most common interface reinforcement details specified for spliced I-girder bridges, the survey participants were asked to identify how the bars are typically detailed in their state/district. More than one answer could be selected. The responses to the survey question are summarized in Table 3.2. The California, Florida, and Washington State DOTs had the most experience in spliced I-girder technology among the survey participants, and these states are thus emphasized in the table. The responses revealed that there is no consensus as to what interface reinforcement detail is most suitable. The implications of the interface reinforcement provided within the splice region is further examined within the experimental program in the following chapters.

Table 3.2: Use of Various Details for Longitudinal Interface Reinforcement

Straight Bars	90-Degree Hooks	180-Degree Hooks	Hairpins	Headed Bars
HI, VA, **WA**	**CA,** GA, NY, NC	**FL,** GA, NC	AZ, **FL,** GA, MA, NY, NC, VA	None

3.3.6 Duct Diameter to Web Width Ratio

The industry survey provided the opportunity to determine the combinations of girder web width, b_w, and duct diameter that have been used in existing spliced I-girder bridges. The combinations of web width and duct diameter that have been specified within each state along with the estimated percent of projects that have used each combination, according to the survey responses, are presented in Table 3.3. The corresponding duct diameter to web width ratios are also presented, and the maximum value is 0.56. Article 5.4.6.2 of AASHTO LRFD (2014) limits the duct diameter to web width ratio to a value of 0.4. This limit has been surpassed in all 10 states where the DOT has recent experience with spliced I-girder technology.

Table 3.3: Combinations of Web Width, b_w, and Duct Diameter

State	Web Width, b_w (in.)	Duct Diameter (in.)	$\frac{\text{Duct Diameter}}{b_w}$	Percent of Projects	Note
Arizona	8	4	0.5	100	---
California	8	4	0.5	30	---
	8	3½	0.44	50	---
	8	3	0.38	20	---
Florida	8 (+/-)	4	0.5 (+/-)	Not Provided	Steel Ducts
	8½	4	0.47		PP* Ducts
	9	4	0.44		PP* Ducts
	7	2⅜	0.34		PE** Oval Ducts
Georgia	9	3.82	0.42	50	---
	12	2 (in pairs)	---	50	---
Hawaii	7⅞	4⅜	0.56	33	---
	8½	4⅝	0.54	33	---
	14	4⅜	0.31	33	---
Massachusetts	7	3	0.43	100	---
New York	8	4	0.5	50	---
	7	3	0.43	50	---
North Carolina	8	3.42	0.43	33	---
	9	3.82	0.42	67	---
Virginia	9	3.7	0.41	50	---
	8	3¼	0.41	50	---
Washington	8	4¼	0.53	100	---

*PP = Polypropylene
**PE = Polyethylene

3.3.7 Location of Splice Regions Relative to Transverse Diaphragms

The next question on the industry survey asked whether the states/districts prefer to locate transverse diaphragms at the same location as the CIP splice regions or if they typically place splice regions away from the diaphragms (refer to Figure 3.5). Locating a transverse diaphragm at the splice region provides benefits such as improved consolidation of concrete due to the extra space within the diaphragm formwork, additional stability during construction, and improved concrete confinement (Castrodale and White, 2004). According to the survey responses, out of the 10 state DOTs having recent experience with spliced I-girder bridges, 7 prefer to locate splice regions at transverse diaphragms. California, Georgia, and Washington, however, prefer to place them away from transverse diaphragms.

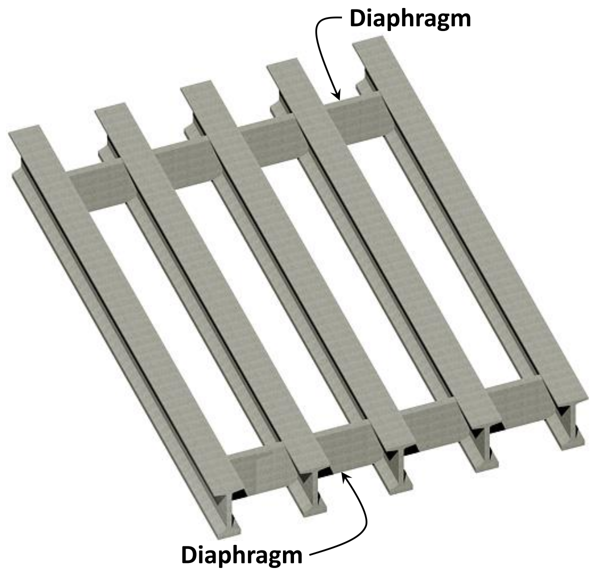


Figure 3.5: Transverse diaphragms

3.3.8 Transverse Width of Splice Region

One of the most critical components for the detailing of spliced I-girders is the geometry of the splice regions. The range of possible splice region geometries were considered by examining both the transverse widths and the longitudinal lengths of splices within existing bridges.

The transverse width of CIP splice regions (i.e., the member cross-section at the splice) is often chosen to match one of the options illustrated in Figure 3.6. Considering the extra space transverse diaphragms may provide, cases in which the splice regions are located away from transverse diaphragms were of primary interest when examining the typical width of the splice according to the industry survey results. The detailing practices in California, Georgia, and Washington were therefore considered.

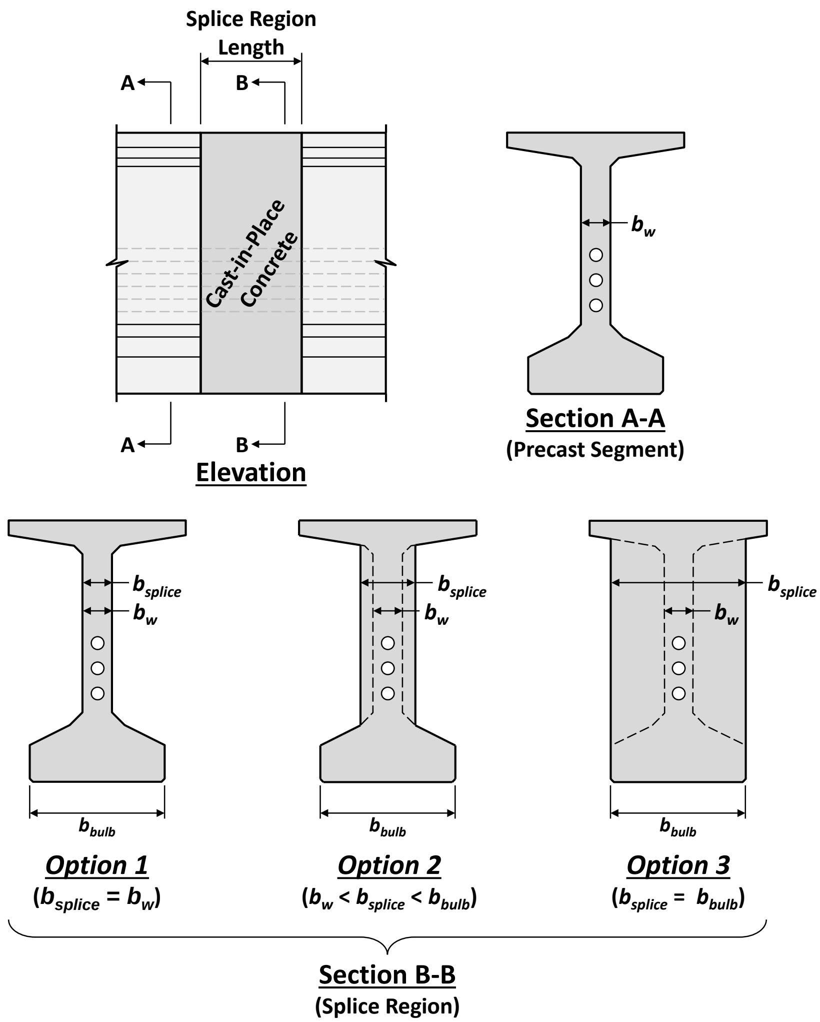


Figure 3.6: Possible options for the transverse width of splice regions

According to the survey responses, the width of the web at the splice region has been typically widened to match the width of the bottom flange in California. The width has been selected to allow ease of forming the splice section and to provide enough space for reinforcement and concrete placement. In contrast, the Washington State DOT has typically maintained a constant cross-section through the splice region in the past. In other words, the splice regions match the shape of the adjacent precast I-girders. The survey responses indicated that the Georgia DOT has only been involved in the design/construction of two spliced girder bridges. One of these bridges, constructed in the early 1990s, contains an atypical splice characterized by a stepped joint. The other bridge, constructed more recently, includes splice regions with a cross-section matching that of the adjacent girders.

3.3.9 Length of Splice Region

The survey participants were asked to provide the values for the lengths of the splice regions that have been specified for existing spliced I-girder bridges. The responses are summarized in Table 3.4. The minimum length that has been specified is 10 in. (with the exception of the 4-in. splice with a stepped joint in Georgia), while the maximum length is 48 in. Considering the typical lengths for each state, the most common value is 24 in.

Table 3.4: Length of Splice Region

State	Minimum Length (in.)	Maximum Length (in.)	Typical Length (in.)
Arizona	---	---	24
California	24	48	24
Florida	18 (+/-)	20 (+/-)	N/A
Georgia	4	30	N/A
Hawaii	24	36	24
Massachusetts	12	14	12
New York	---	---	10
North Carolina	24	Dependent Upon Skew	24
Virginia	---	---	12
Washington	24	Special Cases	24

The primary factors affecting the selected splice region length according to the survey responses include the need for adequate space to place shear reinforcement, splice the post-tensioning ducts, and properly place concrete. Some state DOTs mentioned that the splice region length must allow for the development and lap splicing of longitudinal reinforcement.

3.3.10 Serviceability and Aesthetic Issues

Near the end of the industry survey, the participants were given the opportunity to describe any specific problems they have encountered related to the splice regions of existing bridges. When asked if any serviceability or aesthetic issues have arisen (e.g., cracking, discolored concrete, etc.), the New York State DOT answered that the color of the concrete within the splice region generally does not match the color of the precast girders. All other survey participants answered that no serviceability or aesthetic issues have been observed.

3.3.11 Constructability Issues

Several of the survey responses indicated that constructability issues related to splice regions have been encountered in the field. Most of the problems were associated with either the placement of concrete within the splice regions or the use of temporary supports. Three of the responses (California, New York, and Washington) mentioned concrete consolidation issues at the splice regions, highlighting the importance of proper mixture designs and adequate vibration of the concrete. Three other DOTs (Florida, North Carolina, and Virginia) noted issues with temporary shoring or strong-backs. For example, the Virginia DOT observed cracking at splice regions due to shoring that allowed the pier segments to rotate slightly. Such issues underscore the need for the careful review of construction sequences, falsework submittals, and any documentation related to temporary supports.

3.4 SUPPLEMENTARY MATERIAL

The industry survey participants were also given the opportunity to submit relevant supplementary material with their responses. Several state DOTs provided drawings of existing spliced girder bridges. Some participants also gave access to design guidelines and/or other design-related documents. The supplementary material was reviewed during the development of the splice region details of the test specimens and provided a deeper insight into design and construction procedures that have been successfully implemented.

3.5 SUMMARY

The evaluation of the responses from the industry survey resulted in an enhanced awareness of typical spliced girder design and detailing practices in various states across the U.S. Out of the 25 responses received from DOTs, 10 indicated recent experience in spliced I-girder technology. Survey participants put forth special efforts to provide useful information for the spliced girder research, particularly in regards to CIP splice region details. As described in Chapter 4, the survey responses helped to guide the development of the splice details that were constructed and proof tested for the experimental program.

Chapter 4. Experimental Program

4.1 Overview

The experimental program described in this chapter was conducted to better understand the behavior of spliced I-girders. More specifically, the primary objective was to evaluate the structural performance of cast-in-place (CIP) splice regions. The splice region details of the test specimens were selected based on the industry survey and related supplementary material described in Chapter 3 along with technical input provided by the TxDOT Project Monitoring Committee and a project advisory panel. The experimental program included load tests on two large-scale girder specimens. The tests provide insights into the strength and behavior of spliced girders that are otherwise unavailable within the literature. The design and detailing of the spliced girder specimens are presented in this chapter. The fabrication and testing procedures that were followed for both girders are also outlined.

4.2 Project Advisory Panel

Consultation with knowledgeable professionals with first-hand experience in spliced girder technology is essential to understanding the intricacies involved in the design and construction of spliced I-girder bridges. A project advisory panel was selected to fulfill this need and offer insights and suggestions related to the experimental program. The advisory panel consisted of the followed practitioners:

- Bijan Khaleghi, State Bridge Design Engineer for the Washington State Department of Transportation (WSDOT)
- Steve Seguirant, Vice President and Director of Engineering at Concrete Technology Corporation in Tacoma, Washington

- Christopher White, a senior bridge engineer at Michael Baker Jr., Inc. in Houston, Texas, and coauthor of NCHRP Report 517, *Extending Span Ranges of Precast Prestressed Concrete Girders* (Castrodale and White, 2004)

4.3 SECTION GEOMETRY

The section geometry of the two spliced girder test specimens is illustrated in Figure 4.1. The section was based on the geometry of a Tx62 girder except that all horizontal (i.e., transverse) dimensions were increased by 2 in. The specimens therefore had a web width of 9 in., unlike the standard 7-in. web of TX girders. The increased web width was needed to accommodate ducts with a 4-in. diameter. The end regions of the spliced girder specimens had thickened end blocks for proper anchorage of the post-tensioning tendons (refer to Section 4.7).

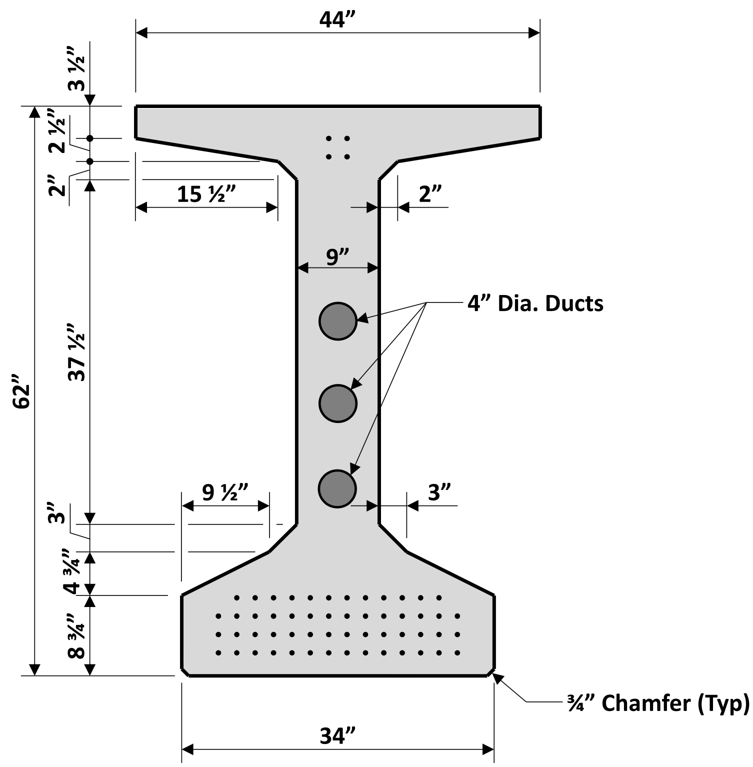


Figure 4.1: Section geometry of spliced girder test specimens

A deck with a thickness of 8 in. was placed on the girder specimens prior to testing. The deck geometry is described in Section 4.13.6.

4.4 SPECIMEN CONFIGURATION

The spliced girder test specimens each consisted of two precast girder segments that were joined at a cast-in-place splice region, as illustrated in Figure 4.2. The short precast segment had a length of 14 ft while the long segment had a length of 34 ft. The CIP splice region was 2-ft long, giving a total specimen length of 50 ft. The geometry of the splice region is further discussed in Sections 4.9.1 and 4.9.2.

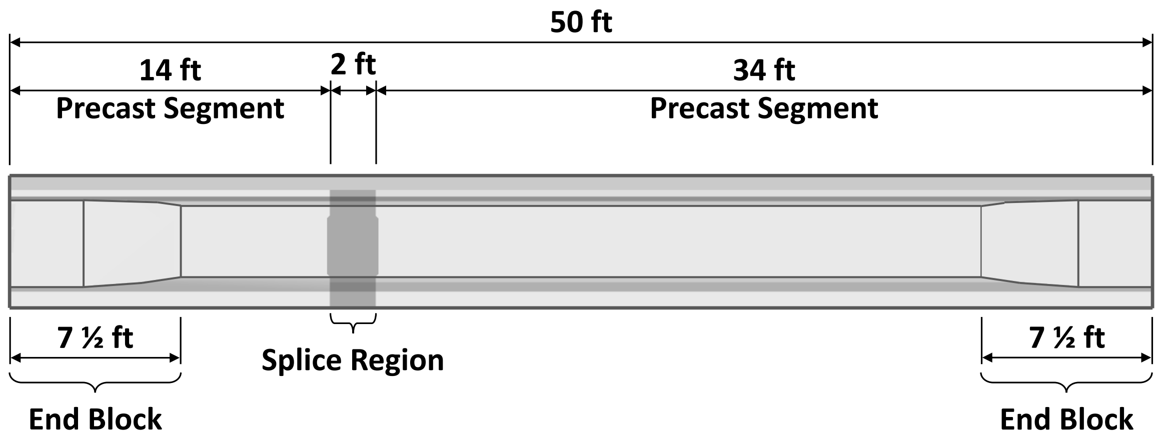


Figure 4.2: Spliced girder specimen configuration

4.5 Pretensioning Strand Layout

The precast segments of the test specimens were pretensioned with 0.5-in. diameter low-relaxation 7-wire prestressing strands with a specified tensile strength, f_{pu}, of 270 ksi (ASTM A416). An identical strand pattern, illustrated in Figure 4.3, was used for all four precast segments fabricated for the testing program. Within the bottom flange, 54 strands were placed in four horizontal rows. The top flange contained 4 strands distributed in two rows. No strands were debonded along the length of the precast segments. Each strand was tensioned to a stress of 202.5 ksi within a tolerance of ±5 percent. The stresses in the extreme fibers of the girder cross-section at prestress transfer were calculated using gross sectional properties. A concrete compressive release strength, f'_{ci}, of 6.0 ksi was specified to satisfy the required stress limits of TxDOT's *Bridge Design Manual – LRFD* (2013).

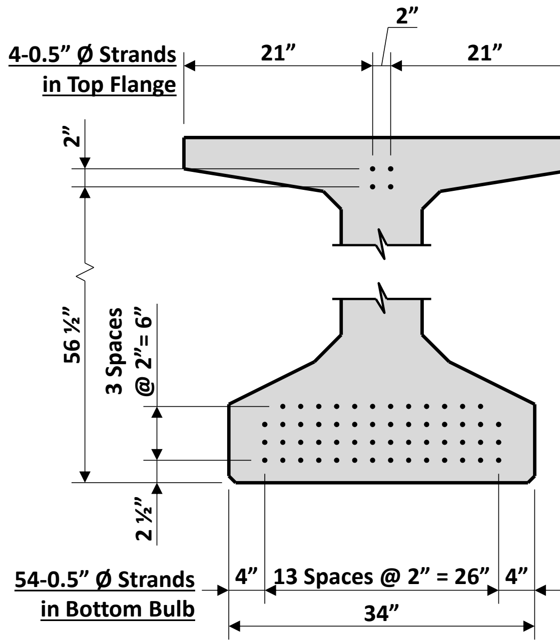


Figure 4.3: Pretensioning strand layout

4.6 End Region Reinforcement (Near Splice Region)

Supplementary vertical reinforcement was placed within the non-thickened end regions of the precast girder segments (i.e., girder ends near the splice region) to provide splitting resistance (see Figure 4.4(a)). The end-region reinforcement satisfied Article 5.10.10.1 of AASHTO LRFD (2014). This provision requires that the vertical reinforcement placed within a distance $h/4$ from the end of the girder (where h is the overall height of the member) be capable of providing a resistance equal to 4 percent of the total prestressing force at transfer without exceeding a stress of 20 ksi. The supplementary reinforcement consisted of straight No. 5 bars paired with the legs of the first three stirrups near the ends of the girder segments. The contribution of these straight bars to the shear strength of the girders was neglected in strength calculations. Confinement reinforcement was also provided in the bottom flange within the girder end regions to satisfy Article 5.10.10.2 of AASHTO LRFD (2014). At the minimum, this provision requires that reinforcement consisting of No. 3 bars spaced at 6.0 in. be provided within a distance of 1.5*d* from the end of the girder. Consistent with standard TX girders, the confinement reinforcement near the splice regions of the test specimens consisted of No. 4 bars spaced at 6.0 in. The details of the reinforcement, however, were slightly different from the bars typically used in standard TX girders. The diagonal extensions of the reinforcement were made longer to control any vertical cracks that may develop under the bottom post-tensioning duct, as illustrated in Figure 4.4(b).

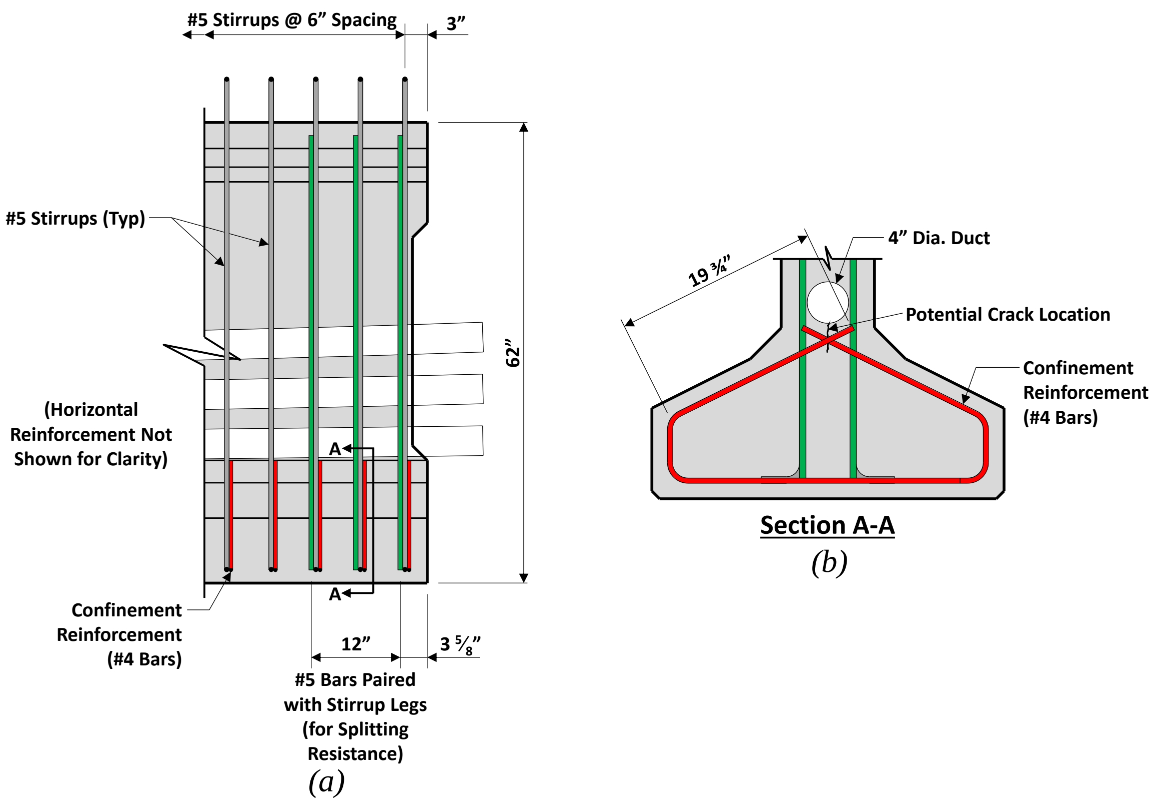


Figure 4.4: End region reinforcement – (a) vertical reinforcement; (b) confinement reinforcement

4.7 END BLOCK DESIGN AND DETAILS

Thickened ends blocks were required at the two ends of the 50-ft long test girders to house the post-tensioning anchorage hardware and allow the post-tensioning force to be properly transferred to the concrete (refer to Figure 4.2). One end of each of the Tx62 precast girder segments was therefore detailed with an end block with a total length of 7 ½ ft. To determine the layout of mild reinforcement within the end blocks, design requirements for both the local and general zones were considered. In accordance with Article 5.10.9.7.3 of AASHTO LRFD (2014), the local anchorage zone reinforcement specified within the end blocks complied with the details provided by the manufacturer of the post-tensioning anchorage hardware. The local zone reinforcement consisted of No. 5 spiral reinforcing bars along with 12-in. by 12-in. closed loops made from No. 5 bars. The general zone was designed using strut-and-tie modeling while satisfying other applicable provisions within AASHTO LRFD. The design and details of the end blocks are provided in Moore (2014). Detailed drawings of the ends blocks of the spliced girder test specimens are presented in Appendix B, and a photograph of one of the corresponding reinforcing cages is shown in Figure 4.5.

Figure 4.5: Reinforcing cage within end block

4.8 Post-Tensioning

4.8.1 Duct Material and Tendon Layout

Three post-tensioning ducts were contained within the web of the test girders and extended the full length of the specimens. The ducts had a diameter of 4 in. Plastic ducts were selected, as opposed to steel ducts, primarily due to the size of the couplers used to join the duct segments together. The relative size of the coupler for a plastic duct with a given nominal diameter is typically larger than the coupler for a steel duct with that same diameter. The use of plastic ducts allowed any localized effects due to the relatively large duct couplers to be identified during testing. Additional information of the specific duct coupler installed within the test girders is provided in Section 4.9.7.

Each of the post-tensioning ducts contained 12 0.6-in. diameter low-relaxation 7-wire prestressing strands with a specified tensile strength, f_{pu}, of 270 ksi (ASTM A416).

The tendons were draped to provide the necessary flexural strength within the test region to ensure the girders exhibited a shear failure. The tendon layout is presented in Figure 4.6. The elevation to the center of each duct (measured from the bottom of the girder) is provided in the figure at 2-ft increments along the length of the test specimens.

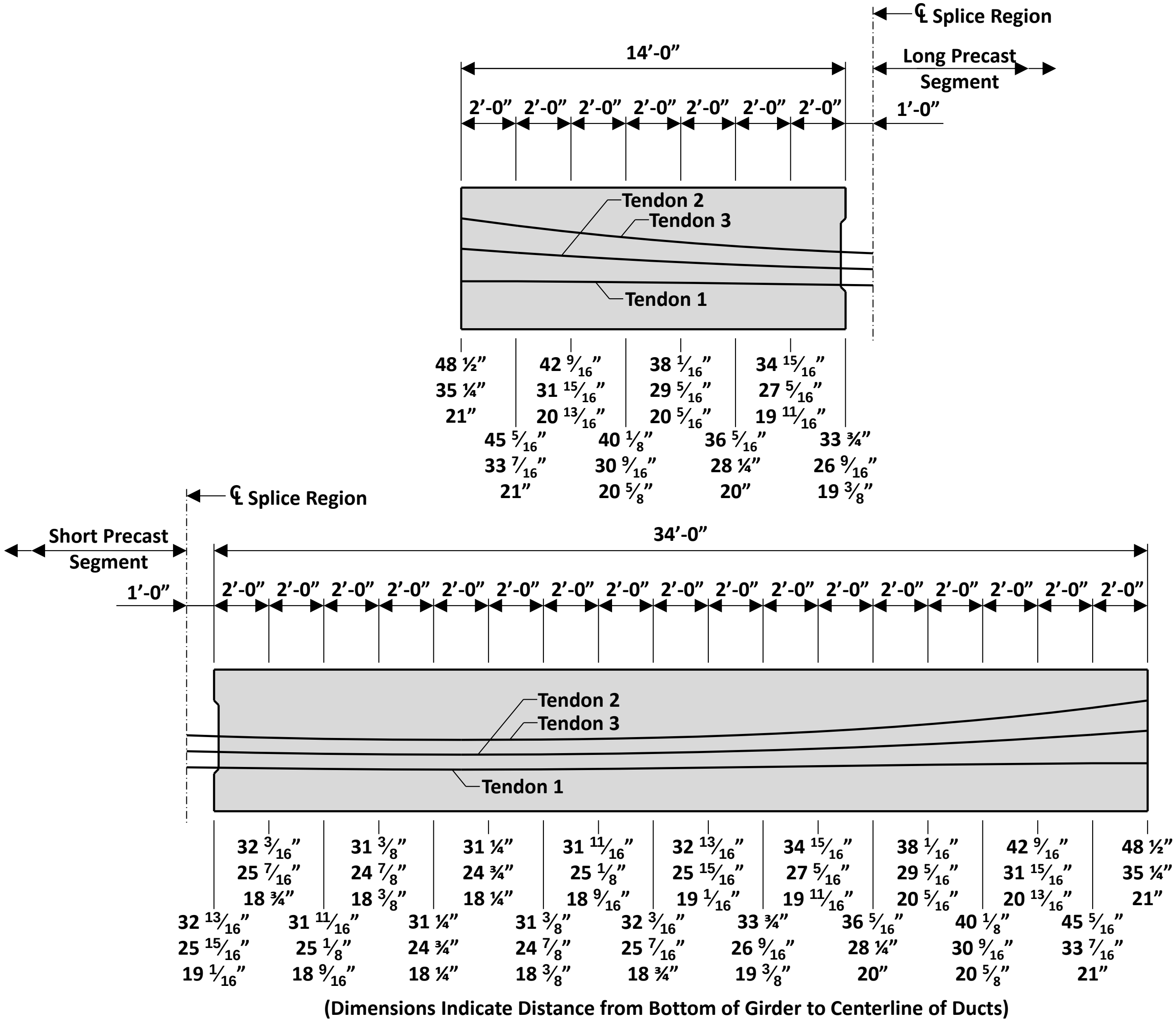


Figure 4.6: Post-tensioning tendon layout

4.8.2 Anchorage

The anchorage hardware installed with each post-tensioning tendon is shown in Figure 4.7. A cast iron multi-plane bearing trumplate was embedded in the concrete at the ends of the girders. Steel anchor heads were later installed on each tendon during the post-tensioning procedures. Information regarding the local zone reinforcement required to be used with the anchorage devices is provided in Section 4.7.

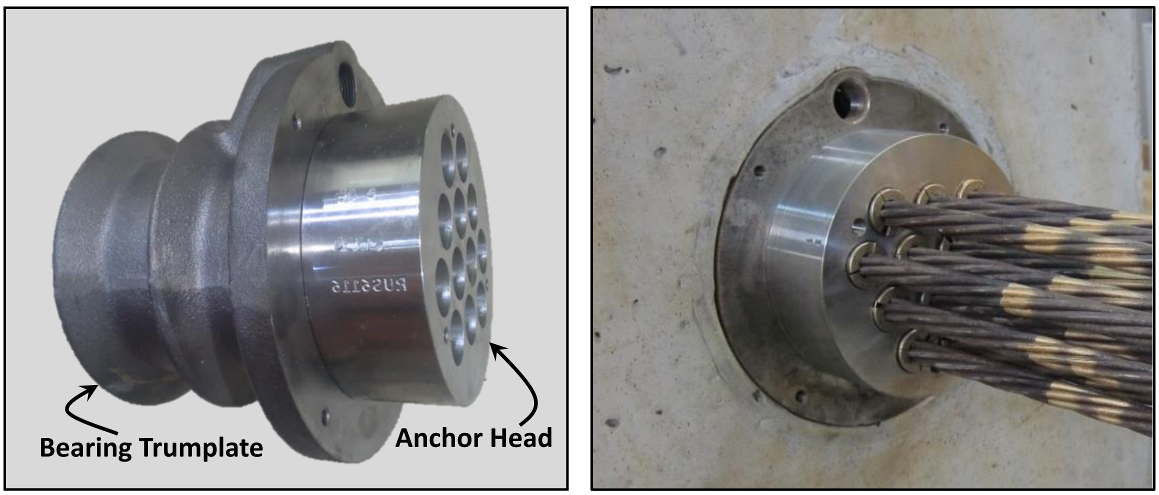


Figure 4.7: Post-tensioning anchorage (adapted from Moore, 2014)

4.9 SPLICE REGION DETAILS

The splice region details of the two test specimens were selected to conduct proof tests that would provide valuable information that could then be directly applied to the design and detailing of actual field structures. The industry survey results as well as input from the project advisory panel and the TxDOT Project Monitoring Committee were all invaluable resources in the development of the details. While designing the splice regions, one of the primary considerations of the details was their simplicity in application. Furthermore, the details were selected to create test specimens representative of existing spliced girder bridges while also considering critical design and construction scenarios.

4.9.1 Length of Splice Region

The length of the splice region measured along the longitudinal axis of the girder was chosen to be 24 in. for both test specimens. This value was primarily based upon the industry survey results, which showed that 24 in. is the most typical splice region length, and discussions with the project advisory panel. A length of 24 in. provides the space needed to place stirrups, splice the post-tensioning ducts, and properly cast concrete. The chosen length also offers the space necessary to accommodate any minor duct misalignment issues within the splice region.

4.9.2 Transverse Width of Splice Region

The transverse width of the splice regions of the testing program (i.e., the member cross-section at the splice) was selected to match the shape of the adjacent precast girder segments. Maintaining a constant cross-section through the splice region gave a worst-case scenario in terms of constructability (e.g., concrete placement). Moreover, it provided the opportunity to study the behavior of a splice region with a restricted cross-sectional area. The findings from the proof tests can therefore be applied to spliced girder bridge designs in which a constant cross-section along the span length is desired for aesthetic reasons. An illustration of the selected splice region geometry is presented in Figure 4.8.

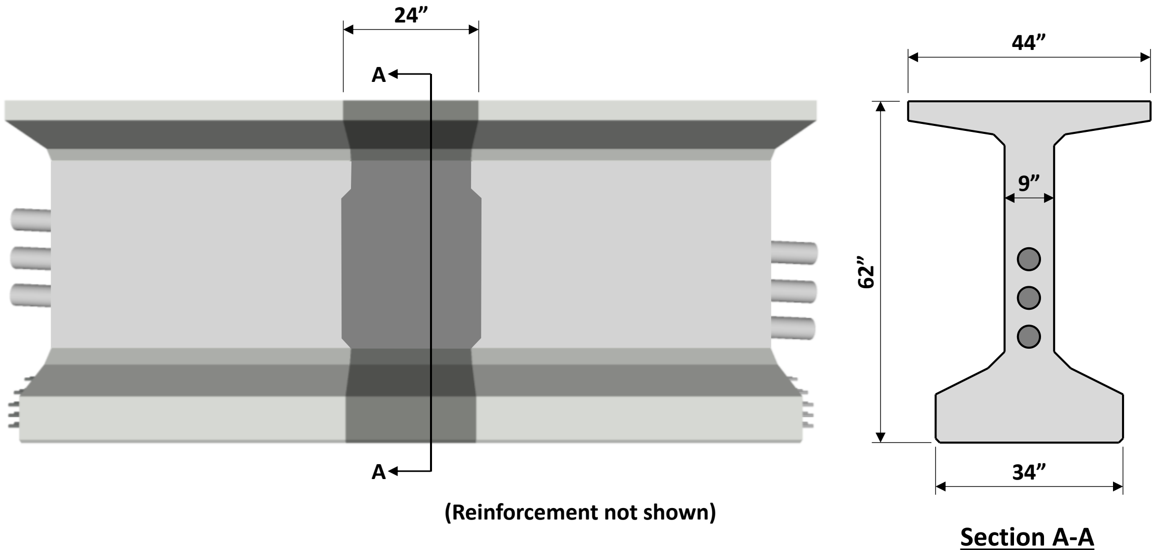


Figure 4.8: Geometry of splice region

4.9.3 Shear Interface Detail

Spliced girders are typically designed with an interface detail at the transitions between the precast segments and the cast-in-place splice regions to aid in shear transfer. A single shear key was chosen as the interface detail for the spliced girder test specimens, as illustrated in Figure 4.9. The shear key had a 2-in. inset and was contained within the web of the girder. The selection of a single shear key was based on its successful application in existing spliced girder bridges as indicated by the industry survey (refer to Section 3.3.4). Moreover, the detail is simple and the required formwork is relatively easy to fabricate.

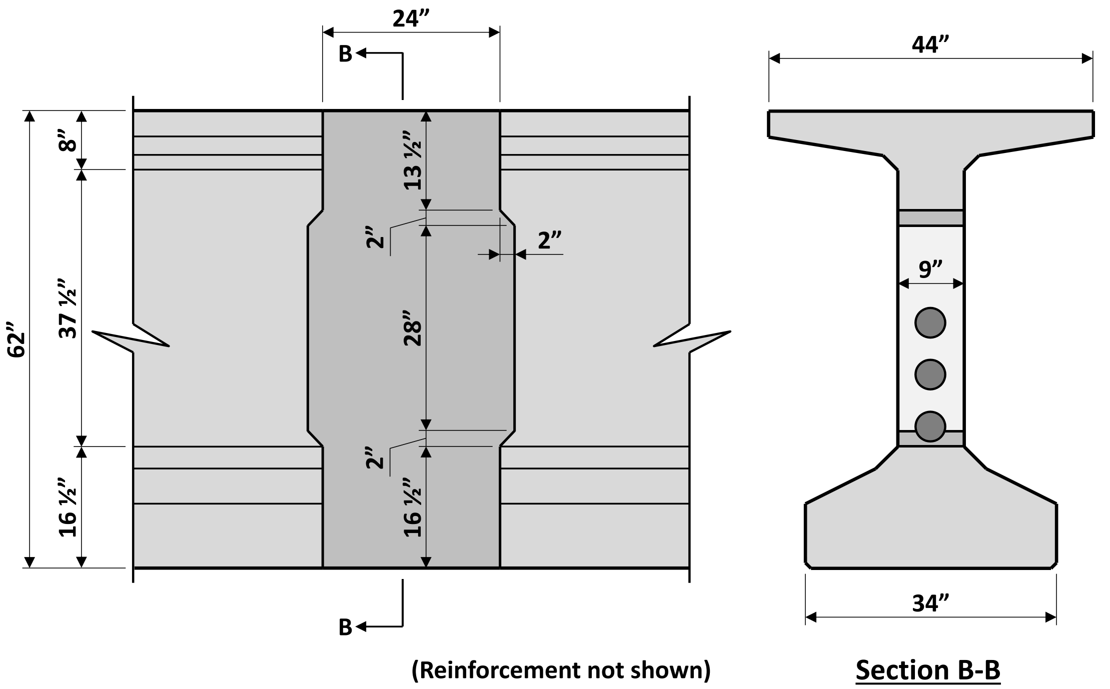


Figure 4.9: Shear key detail

4.9.4 Longitudinal Interface Reinforcement

The effect of the mild longitudinal reinforcement crossing the interface between the precast girder segments and the splice region was one of the primary interests of the testing program. Straight bars were selected to provide a simple detail that would lead to reduced congestion in the splice region as compared to some other possible bar detailing options.

The interface reinforcement details of the two test specimens are illustrated in Figures 4.10 and 4.11. The amount of interface reinforcement was varied between the two specimens to determine the effect of the bars on the behavior of the splice region. Within the bottom flanges of the test girders, 6 No. 4 bars (A_s = 1.2 in.^{2}) extended from each precast segment into the splice region of the first specimen, while 8 No. 6 bars (A_s = 3.52 in.^{2}) extended into the splice region of the second specimen. The second test girder therefore contained approximately 3 times the area of interface reinforcement within its bottom flange at the splice region compared to the first girder. Both test specimens contained No. 4 interface bars along the height of the web, as shown in Figures 4.10 and 4.11. Within the top flange, 6 No. 4 bars extended from each precast girder segment of the first test specimen, and 6 No. 5 bars extended from each precast segment of the second specimen. All of the interface reinforcement was embedded 24 in. into the precast segments, and each bar extended 21 in. into the splice region.

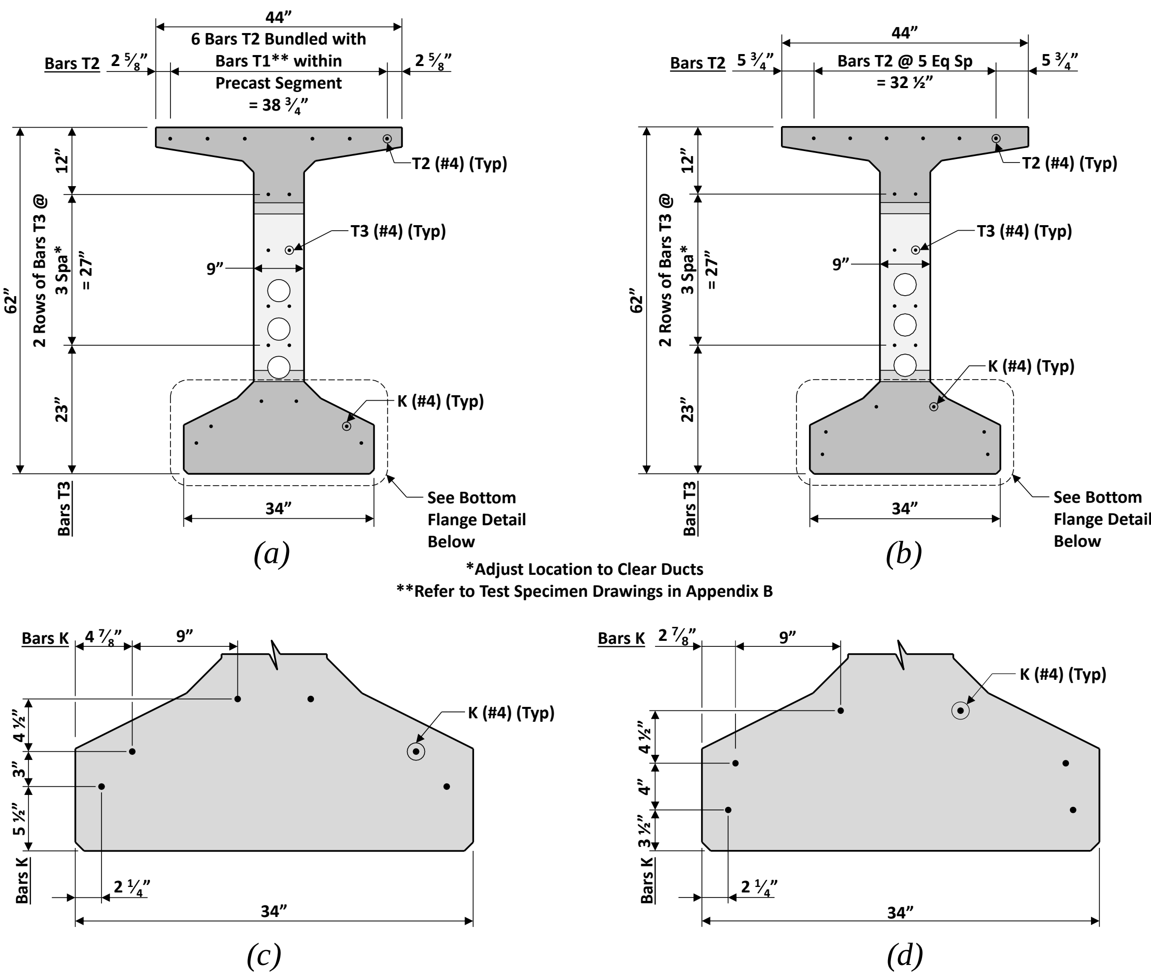


Figure 4.10: Longitudinal interface reinforcement of Test Girder 1 – (a) end of long precast segment; (b) end of short precast segment; (c) flange detail of long precast segment; (d) flange detail of short precast segment

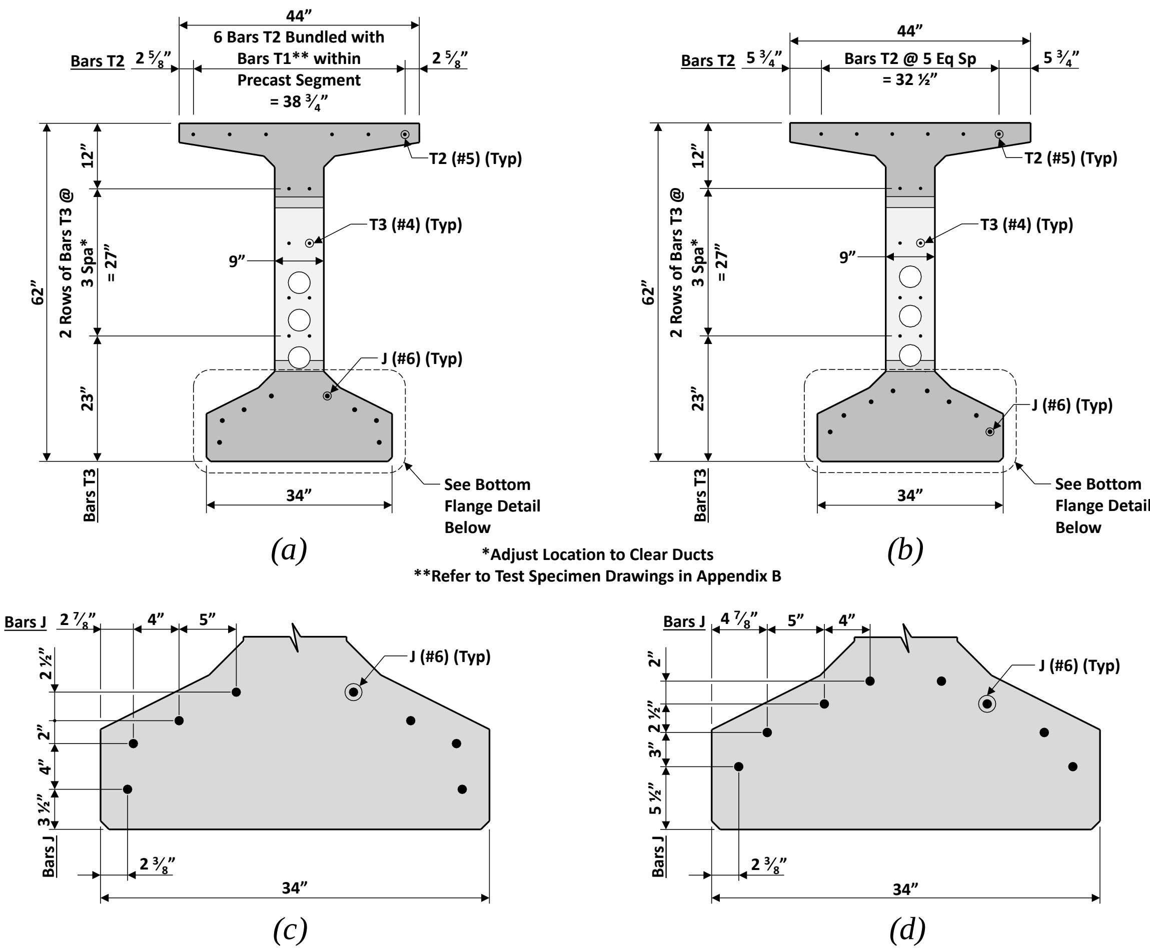


Figure 4.11: Longitudinal interface reinforcement of Test Girder 2– (a) end of long precast segment; (b) end of short precast segment; (c) flange detail of long precast segment; (d) flange detail of short precast segment

All four precast girder segments (i.e., two precast segments for each of the test specimens) were fabricated during the same period due to scheduling restrictions with the concrete precaster. To provide flexibility with the interface reinforcement details of the two test specimens, each precast segment was fabricated with both the No. 4 and No. 6 bars extending from the bottom flanges. The unwanted interface bars were later cut at the surface of the precast segments. For example, the No. 4 bars extending from the bottom flange of the girder segments for the second test specimen were cut off to leave only the No. 6 bars crossing into the splice region.

After fabrication of the precast girder segments, the pretensioned strands extended from both the top and bottom flanges at the beam ends. To avoid additional congestion within the splice region, the pretensioned strands were cut within approximately 3 in. of the surface of the girder segments. Although allowing the strands to extend farther into the splice region would have provided additional steel that may have had a beneficial effect on the performance of the test girders, the risk of concrete consolidation issues due to the added congestion of the strands was determined to outweigh any possible benefits (refer to the mock-up cast described in Section 4.11.2).

4.9.5 Duct Diameter to Web Width Ratio

Ducts with a 4-in. diameter were contained within the 9-in. webs of the test specimens, giving a duct diameter to web width ratio of 0.44. Considering the results of the industry survey presented in Section 3.3.6, a value of 0.44 is within the range of typical duct diameter to web width ratios of existing spliced girder bridges. Furthermore, the ratio is slightly greater than the AASHTO LRFD (2014) limit of 0.4, a value often exceeded in the field.

4.9.6 Shear and Transverse Reinforcement within the Splice Region

The shear and transverse reinforcement within the splice regions of the test specimens was essentially a continuation of the reinforcement provided within the adjacent precast segments, as illustrated in Figure 4.12. The 6-in. spacing of No. 5 stirrups (Bars R) within the girder segments was continued through the splice region. The No. 3 bars (Bars A) that were provided as transverse reinforcement within the top flange of the precast segments were also included in the splice region as shown in Figure 4.12.

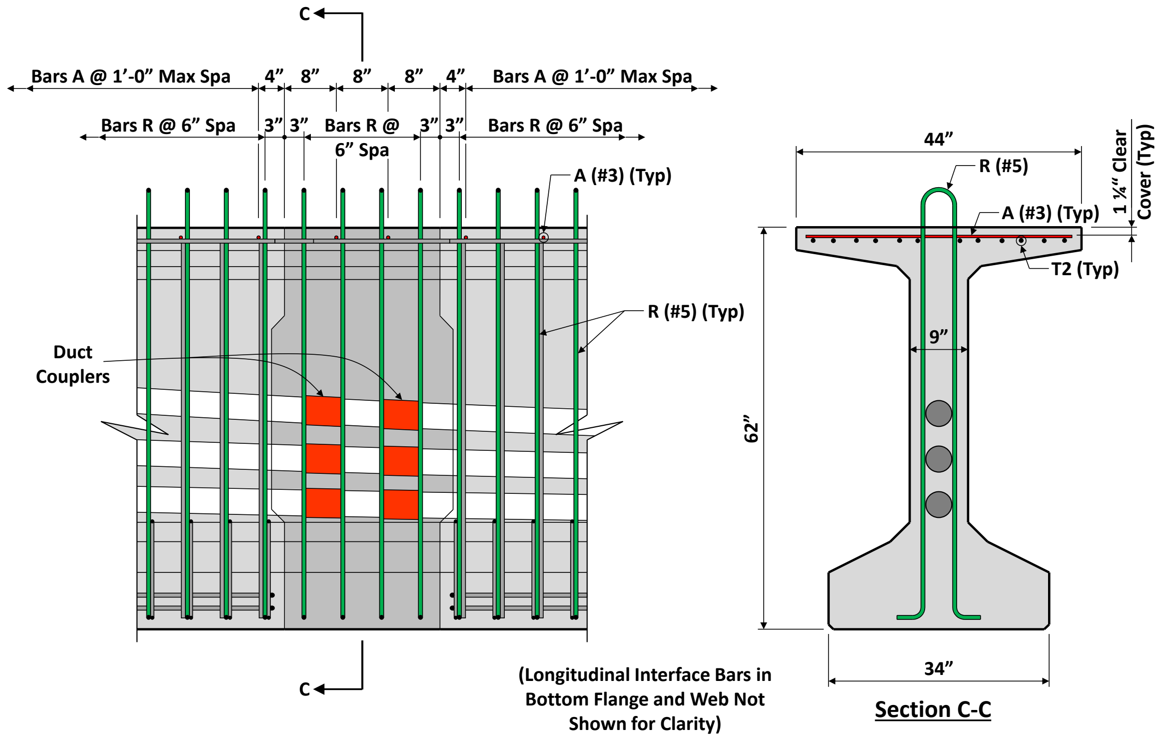


Figure 4.12: Shear reinforcement (Bars R) and transverse reinforcement (Bars A) within the splice region

4.9.7 Duct Coupling Detail

The manner in which the post-tensioning ducts are coupled together within the splice regions is typically included in the detailed drawings of spliced girder bridges. The ducts are generally coupled using either a single coupler at the center of the splice region or by using two couplers with a short duct segment in the middle. Strength and constructability were both major factors when specifying the duct coupling detail within the splice regions of the test specimens. Considering these two factors, the detail presented in Figure 4.13(a) was developed. If the relatively large plastic duct couplers have a detrimental effect on the shear behavior of the specimens (refer to Section 4.8.1), the chosen detail with two couplers may result in a more critical (i.e., worst-case) scenario than the use of only one coupler. Furthermore, the detail with two couplers better accommodates minor misalignment of the ducts extending from the precast segments, simplifying construction.

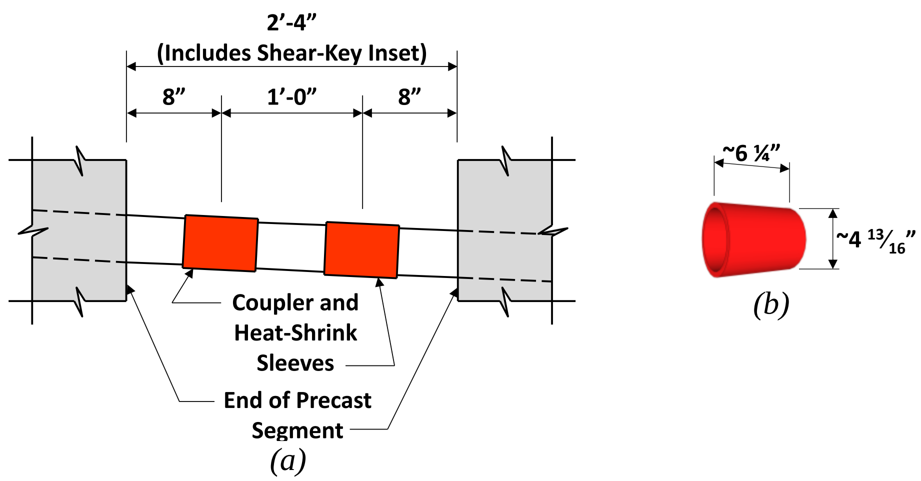


Figure 4.13: Duct coupling – (a) construction detail; (b) coupler dimensions

The duct couplers used within the test girders are slip-on couplers, as opposed to snap-on couplers. The approximate dimensions of the couplers are provided in Figure 4.13(b). Heat-shrink sleeves were used to seal the ends of each duct coupler (refer to Figure 4.19(d)).

4.10 Precast Concrete Mixture

The four precast girder segments of the testing program were fabricated offsite at a precast/prestressed concrete yard. The precasting plant typically used a standard concrete mixture for casting TX girders. This same mixture was also used to fabricate the four precast segments. The details of the concrete mixture design are provided in Table 4.1. It was classified as a self-consolidating concrete (SCC). The mixture included over 900 lbs of cementitious material per cubic yard of concrete and contained ½-in. (TxDOT Grade 6) river gravel as the coarse aggregate.

Table 4.1: Precast Concrete Mixture Design

Material	Details	Design Quantity	Units
Cementitious Material	Type III Cement	663	lb/yd^3 concrete
	Class F Fly Ash	271	
Coarse Aggregate	River Gravel (1/2" Nominal)	1,555	
Fine Aggregate	Sand (F.M. = 2.7)	1,222	
Water	---	269	
Admixtures	High-Range Water Reducer	5.50	oz/cwt
	Water Reducer/Retarder	2.50	
	Corrosion Inhibiter	41.15	
	Viscosity Modifier	2.78	

4.11 Splice Region Concrete

4.11.1 Mixture Design

The mixture for the splice region concrete was designed to meet requirements for both strength and workability. Any undesired effects caused by two drastically different strengths between the precast concrete and the cast-in-place splice region concrete were avoided. Given the high strength of the concrete used for the precast segments (refer to Section 4.17), a mixture that would result in a relatively high concrete compressive strength within the splice regions of the test specimens was selected. At the same time, the splice region concrete was ensured to be a mixture that would be readily available in the field from a local ready-mix supplier.

In addition to the concrete strength, the workability of the mixture had to be suitable to allow the concrete to flow into the relatively congested splice region without resulting in any consolidation problems. A mock-up cast, described in Section 4.11.2 below, was conducted to ensure the proper workability of the chosen mixture.

The mixture design presented in Table 4.2 was selected after making certain that the desired strength could be achieved and that concrete consolidation issues would be avoided. The chosen mixture had 700 lbs of cementitious material per cubic yard of concrete and contained 1-in. (TxDOT Grade 4) river gravel as the coarse aggregate. The target slump of the mixture was 8.0 in.

Table 4.2: Splice Region Concrete Mixture Design

Material	Details	Design Quantity	Units
Cementitious Material	Type I/II Cement	525	lb/yd^3 concrete
	Class F Fly Ash	175	
Coarse Aggregate	River Gravel (1” Nominal)	1,880	
Fine Aggregate	Sand	1,221	
Water	---	233	
Admixtures	High-Range Water Reducer	5.5	oz/cwt
	Water Reducer/Retarder	2.0 to 3.0	

4.11.2 Mock-Up Cast: Findings and Solutions

The casting of a splice region mock-up was considered to be essential to ensuring that the proper concrete strength would be reached within the splice region of the test specimens and that no consolidation issues would arise. For the mock-up cast, formwork was constructed in the shape of the 2-ft long splice region, as shown in Figure 4.14. Transparent polycarbonate sheets were used to form the I-shaped cross-section to allow the concrete placement to be observed during casting. A single internal vibrator with a ¾-in. diameter head was used to consolidate the concrete. From his location, the operator of the internal vibrator was unable to observe the concrete through the polycarbonate sheeting, similar to actual field conditions.

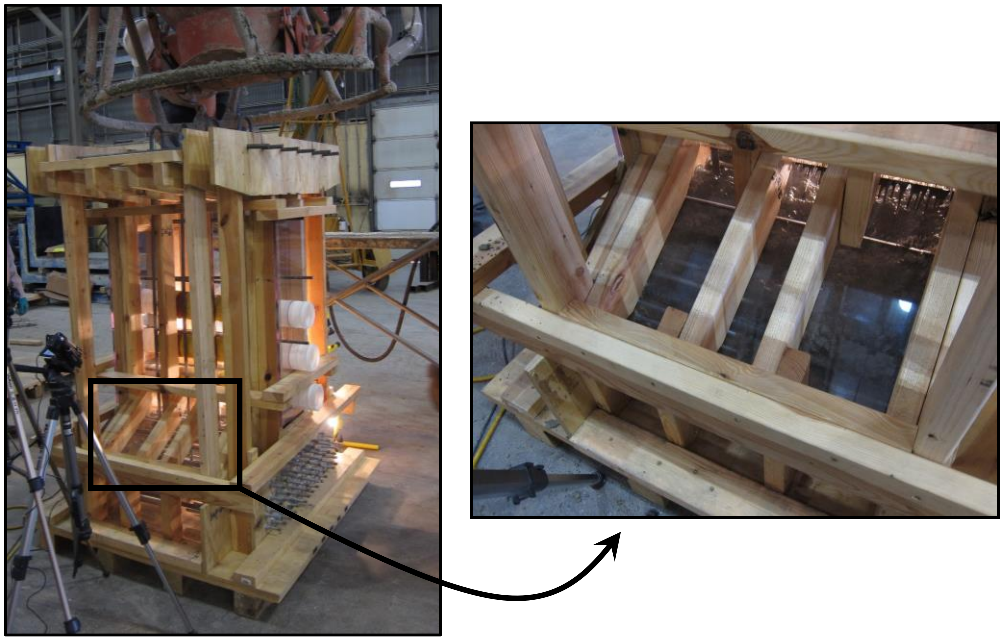


Figure 4.14: Formwork for splice region mock-up cast

After allowing the concrete to cure for several days, the forms were removed. Consolidation issues were observed within the bottom flange of the mock-up specimen as shown in Figure 4.15. No other problems, however, were noted along the height of the specimen. Based on these observations, three measures were taken to ensure that similar consolidation issues did not reoccur when casting the splice regions of the test girders. First, the pretensioned strands extending from the precast segments into the splice region were cut within approximately 3 in. of the surface of the girder segments, as described in Section 4.9.4. The mock-up specimen simulated the strands extending 10 in. into the splice region. Second, external form vibrators were used to ensure the concrete consolidated properly within the bottom flange of the splice region (refer to Section 4.13.3). Lastly, a row of small air holes ($^{5}/_{64}$-in. diameter) were provided along the bottom of the side forms used for the splice regions of the test specimens to allow any trapped air to escape from the bottom flange during casting. The details of the longitudinal interface reinforcement were also updated after the mock-up cast. This change in the details, however, is believed to have had a small effect, if any, on the consolidation of the concrete within the splice region.

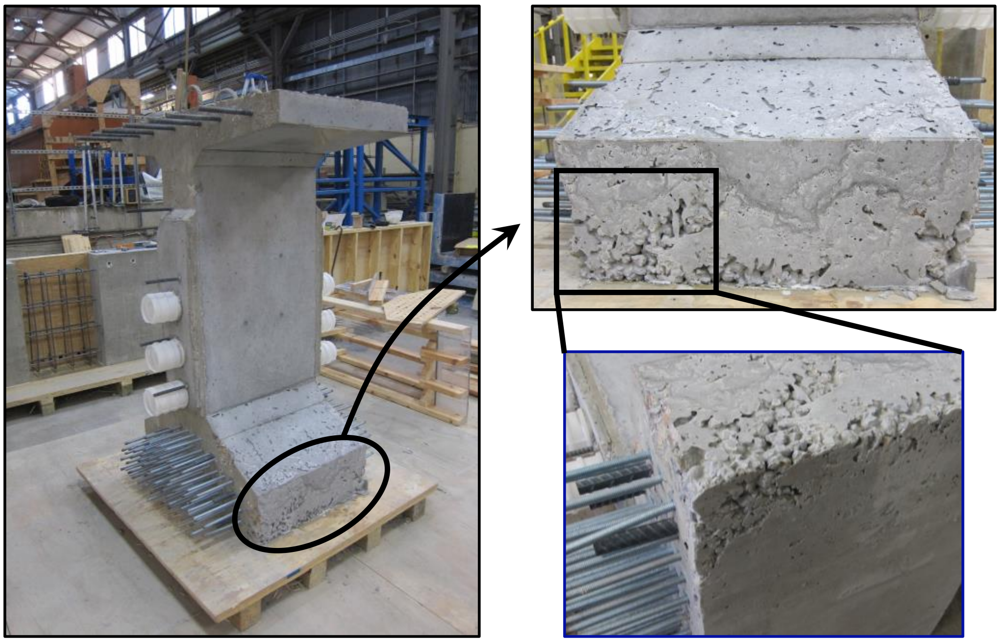


Figure 4.15: Consolidation issues of splice region mock-up cast

The concrete mixture used for the mock-up cast achieved a compressive strength adequate for the splice regions of the test girders. A similar mixture was therefore used for the two splice region casts (refer to Section 4.11.1).

4.12 Deck Concrete Mixture

The concrete mixture design used for the 8-in. thick deck placed on the first test girder specimen is provided in Table 4.3. The mixture had a water-cement ratio of 0.30 and contained $\frac{3}{8}$-in. (TxDOT Grade 7) crushed limestone as the coarse aggregate. A different mixture was used for the deck of the second girder specimen, as presented in Table 4.4, to expedite the experimental program by ensuring adequate strength was reached at an early age. The water-cement ratio was decreased compared to the previous mixture and a hydration controlling admixture was used to enhance the strength gain within the 2 weeks between casting the deck and testing the girder specimen.

Table 4.3: Deck Concrete Mixture Design – Test Girder 1

Material	Details	Design Quantity	Units
Cementitious Material	Type I/II Cement	592	lb/yd^3 concrete
	Class F Fly Ash	200	
Coarse Aggregate	Crushed Limestone (3/8" Nominal)	1,720	
Fine Aggregate	Sand	1,358	
Water	---	238	
Admixtures	High-Range Water Reducer	6.0	oz/cwt
	Water Reducer/Retarder	1.0	

Table 4.4: Deck Concrete Mixture Design – Test Girder 2

Material	Details	Design Quantity	Units
Cementitious Material	Type I/II Cement	658	lb/yd^3 concrete
	Class F Fly Ash	282	
Coarse Aggregate	Crushed Limestone (3/8" Nominal)	1,750	
Fine Aggregate	Sand	1,168	
Water	---	250	
Admixtures	High-Range Water Reducer	6.5	oz/cwt
	Hydration Stabilizer	1.5	

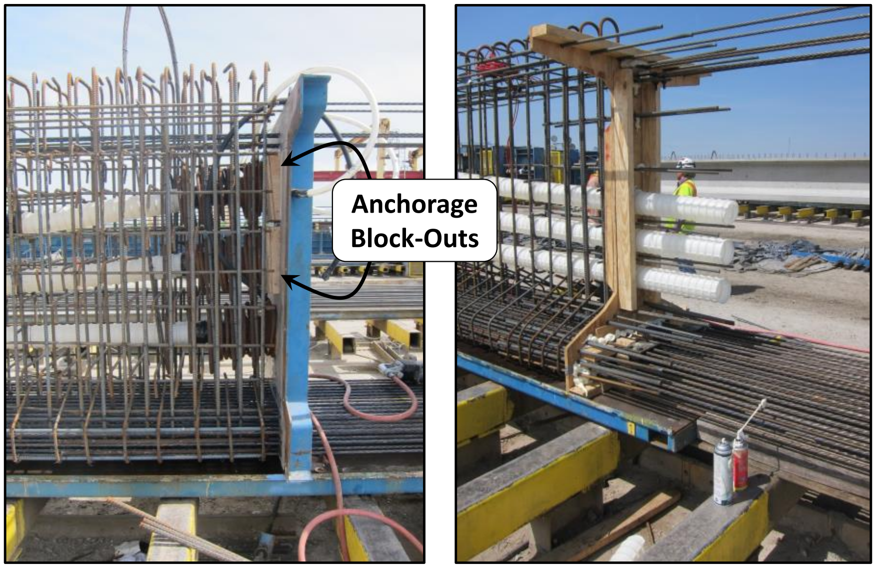
4.13 TEST SPECIMEN FABRICATION

The fabrication and preparation of each spliced girder test specimen consisted of several steps that are outlined in this section. Many of the procedures are similar to those described in Moore (2014) for the monolithic post-tensioned specimens tested as part of the spliced girder research program. Several aspects, however, are unique to the specimens fabricated to study CIP splice region behavior.

4.13.1 Fabrication of Precast Segments

The precast segments of the two test girders were fabricated offsite at a precast/prestressed concrete yard. For each 50-ft long girder test specimen, both precast segments (i.e., the 34-ft long segment and the 14-ft long segment) were cast at the same time. Prior to assembling the reinforcing cages of the precast segments, the pretensioning strands were extended the full length of the prestressing bed. Each strand was then individually stressed to a value of $0.75f_{pu}$, or 202.5 ksi, within a tolerance of ±5 percent.

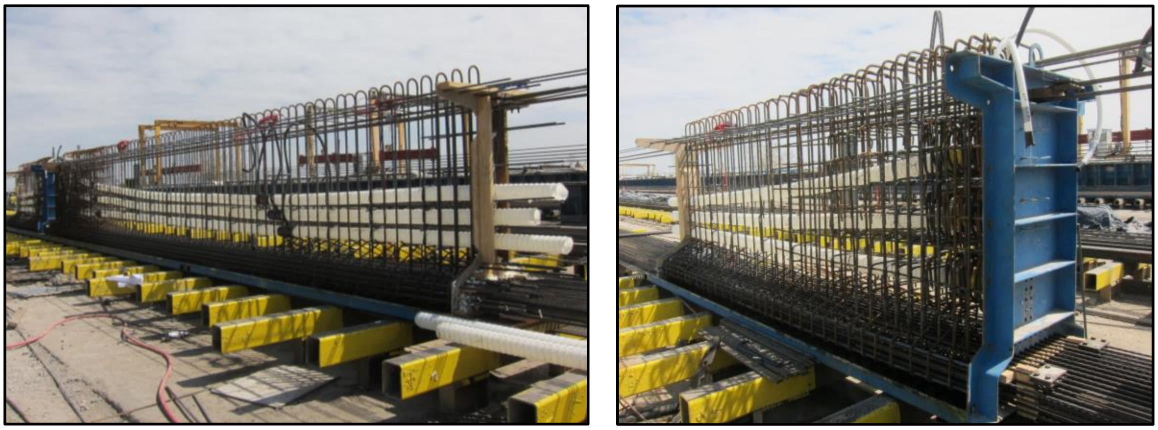
Each precast girder segment had a steel end form placed at the thickened end block. To accommodate the angle of the anchorages of the top two post-tensioning tendons, block-outs were installed with the anchorage hardware as shown in Figure 4.16(a). At the ends of the precast segments that were later to be spliced together, wooden end forms were installed (refer to Figure 4.16(b)). Holes were cut through the wooden end forms at the specified locations of the longitudinal interface reinforcement that was to extend into the splice region.



(a) *(b)*

Figure 4.16: End forms of precast girder segments – (a) at thickened end block; (b) at end to be spliced

Once much of the mild reinforcement was in place, the ducts for the three post-tensioning tendons were installed. The locations of the ducts were then adjusted to match the tendon profiles presented in Section 4.8.1. The ducts were supported at a maximum spacing of 2 ft in accordance with the *Specification for Grouting of Post-Tensioned Structures* (PTI M55.1-12) and the *AASHTO LRFD Bridge Construction Specifications* (2010). After the remaining mild reinforcement was placed, the positions of the ducts were checked before the side forms were installed. The completed reinforcing cages for a set of the precast girder segments are shown in Figure 4.17.



(a) *(b)*

Figure 4.17: Completed reinforcing cages of precast girder segments – (a) long precast segment; (b) short precast segment

The segments were cast with the self-consolidating concrete mixture presented in Section 4.10. Considering the congestion introduced by the reinforcing cage and post-tensioning ducts, an external vibrator was used along one of the side forms of both segments to ensure proper concrete consolidation through the depth of the girders, especially around and below the post-tensioning ducts and within the thickened end blocks. An internal concrete vibrator was also used to ensure satisfactory consolidation.

After casting was complete, the girder segments were covered and left undisturbed until the concrete reached its specified compressive release strength, f'_{ci}, of 6.0 ksi. After the concrete achieved the required strength, the side forms could be detached. With the side forms removed, the stress in all the pretensioned strands was slowly released simultaneously, transferring the pretensioning force to the girder segments. Lastly, the strands were flame cut, and the girder segments were stored at the precast/prestressed concrete yard until transported to the laboratory for further preparation.

4.13.2 Preparation of Precast Segments

Prior to splicing the two precast segments of each test girder, preparatory work was performed on the girder ends located at the splice region. After the precast segments were transported to the laboratory (Figure 4.18(a)) and the wooden end forms were removed, the following steps were completed to prepare for the splicing operation:

(i) Cut pretensioned strands:

The pretensioned strands extending from the top and bottom flanges of the precast segments were cut within approximately 3 in. of the girder faces as described in Section 4.9.4 (Figure 4.18(b)).

(ii) Trim post-tensioning ducts:

The three post-tensioning ducts extending from each precast segment were trimmed at approximately 8 in. from the girder faces in accordance with the detail of Figure 4.13 (see Figure 4.18(c)).

(iii) Cut longitudinal interface reinforcement:

As described in Section 4.9.4, all four precast segments were fabricated with No. 4 and No. 6 longitudinal interface bars extending from the bottom flanges. The interface bars not included in the details of each test girder were therefore cut at the faces of the girder segments to match the reinforcement layouts presented in Figures 4.10 and 4.11.

(iv) Install strain gauges:

Prior to moving the girder segments into their final positions for the splicing operation, foil strain gauges were installed on the interface bars as described in Section 4.14.2 below.

(a)

(b)

(c)

Figure 4.18: Preparing precast segments – (a) transporting girder segments to laboratory; (b) cutting pretensioned strands and interface reinforcement; (c) trimming ducts

4.13.3 Splicing Procedure

Several steps were required to splice the precast girder segments together, including placing the segments in their proper positions and preparing the splice region for casting. The splicing procedure is detailed below and each step is shown for the first test specimen in Figures 4.19 and 4.20.

(i) <u>Place girder segments into position for splicing:</u>

Three concrete pedestals, shown in Figure 4.19(a), were fabricated and moved into position to support the two girder segments of each test specimen. The girder segments were placed onto the pedestals and separated by a 2-ft long gap as shown in Figure 4.19(b). The ends of the segments to be spliced together were supported on the 5-ft long center pedestal. The splice region was later cast directly on this center pedestal.

(ii) Verify proper alignment and placement of the girder segments:

Once the precast girder segments were positioned on the concrete pedestals, the placement of the segments was checked and the vertical and transverse alignment at the splice region was verified (see Figure 4.19(c)). If required, metal shims were used at the supports to aid in aligning the girder segments at the splice region.

(iii) Couple ducts:

After the girder segments were placed in their final positions, the ducts extending into the splice region were coupled as shown in Figure 4.19(d). After the duct couplers were moved into their proper positions as detailed in Figure 4.13(a), heat-shrink sleeves were used to seal the ends of each coupler.

(iv) Splice longitudinal interface reinforcement:

Contact lap splices were used to provide continuity to the longitudinal interface bars extending from the webs of the girder segments. These No. 4 bars were tied together as shown in Figure 4.19(e).

(v) Place shear and transverse reinforcement:

Four No. 5 stirrups (Bars R) spaced at 6 in. were placed within the splice region as detailed in Figure 4.12. The transverse reinforcement (Bars A) were then tied to the longitudinal interface bars extending into the top flange of the splice region (see Figure 4.19(f)).

(vi) Install side forms:

Side forms for casting the splice region were fabricated at the laboratory from hollow structural steel sections. Once all preparations of the splice region were completed as shown in Figure 4.20(a), the side forms were installed onto the girder segments (see Figure 4.20(b)). Threaded rods extended between each side-form piece to clamp the forms to the girder segments.

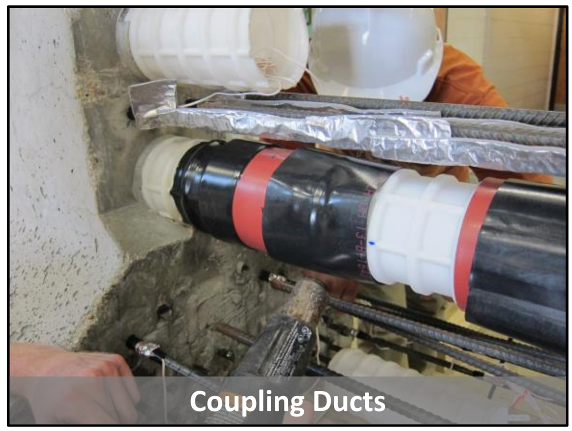
(vii) Cast the splice region:

The final step in the splicing procedure was casting the splice region, shown in Figure 4.20(c). In addition to the use of an internal vibrator with a ¾-in. diameter head, external vibrators were installed on each side form to aid in consolidating the concrete. The bottom flange of the splice region was formed with transparent polycarbonate as shown in Figure 4.20(d) in order to ensure concrete was placed properly during the cast.

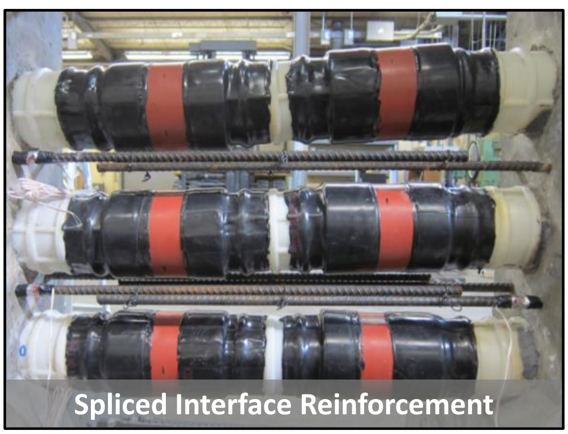
(a)

(b)

(c)



(d)



(e)

(f)

Figure 4.19: Sequence of steps to prepare the splice region – (a) concrete pedestals; (b) placing girder segments; (c) verifying girder placement; (d) coupling ducts; (e) spliced interface reinforcement; (f) shear and transverse reinforcement

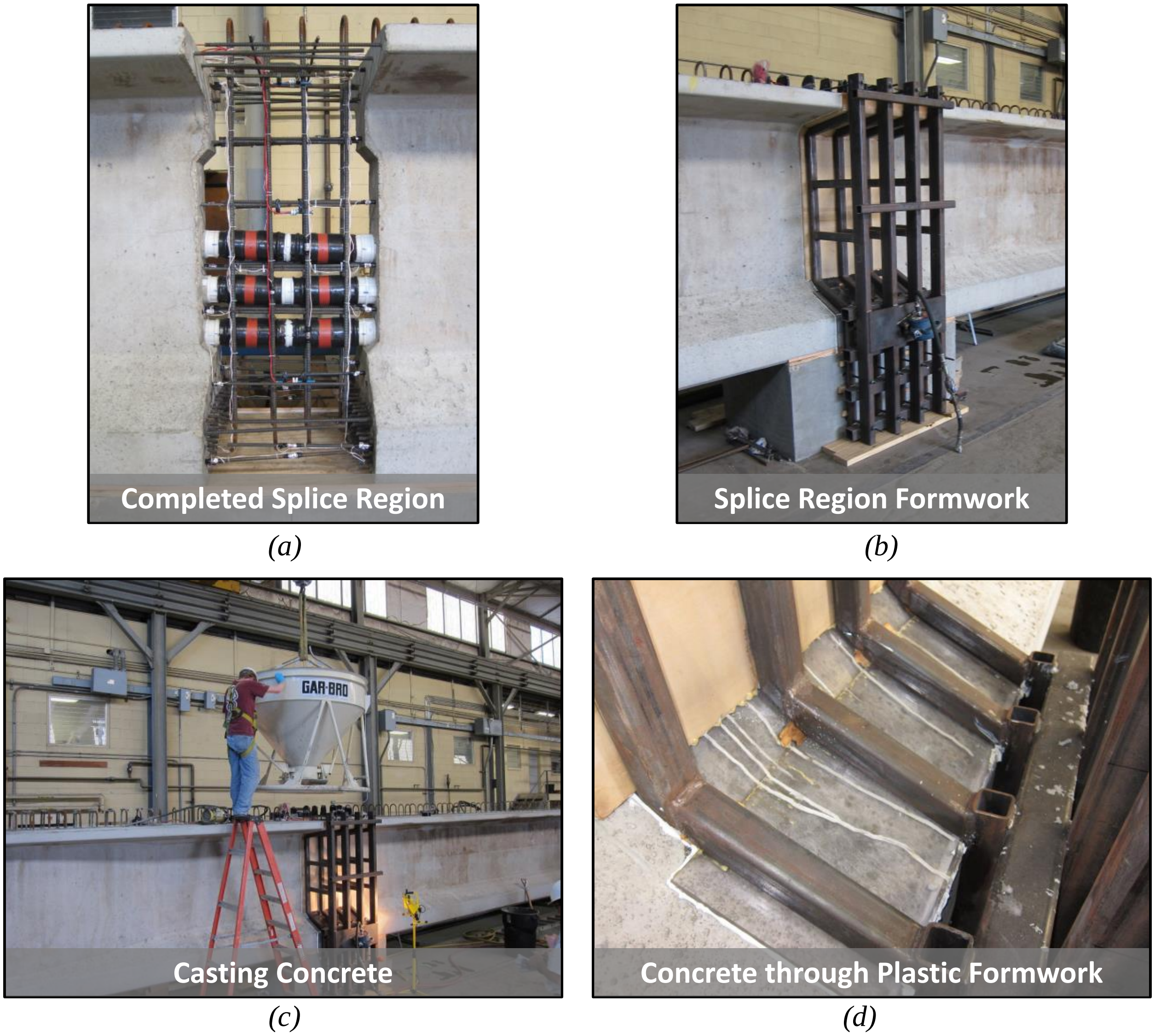


Figure 4.20: Casting the splice region – (a) completed splice region; (b) splice region formwork; (c) casting concrete; (d) concrete through plastic formwork

4.13.4 Post-Tensioning Procedure

After the splice region concrete reached the required strength, the three tendons were post-tensioned. A jacking stress of 212.6 ksi was chosen for the tendons. This value is consistent with the post-tensioning jacking stress of the monolithic girders tested during the first phase of the spliced girder research program and is within the range of the jacking stress specified for field structures. Considering the close spacing of the tendons along with their draped profiles, the post-tensioning sequence was governed by the desire to prevent any risk of the break-through of one tendon into the duct located immediately above that tendon. To eliminate this risk, each tendon of the girder specimen was post-tensioned and then grouted before repeating the procedure for the next tendon. Furthermore, the top tendon was post-tensioned and grouted first, followed by the middle and then the bottom tendons. It should be noted that all post-tensioning operations were completed prior to placing the deck on the test girders. With a deck, the girders would have exceeded the weight limits of the gantry cranes in the laboratory that were used to move the specimens to the strong floor for testing. Moreover, to simplify future deck replacement, designers may choose to specify that all tendons of a spliced girder bridge be post-tensioned prior to casting the deck (refer to Section 1.2.2). The procedure below describes the post-tensioning procedure that was followed for each tendon of the spliced girder specimens. Each tendon was stressed from the same end of the 50-ft long girder.

(i) Insert strands into the post-tensioning duct:

Twelve prestressing strands were first bundled and then manually inserted into the post-tensioning duct. Strand extensions necessary to complete the post-tensioning operation were left at each end of the girder.

(ii) Install anchor heads and set wedges:

An anchor head (refer to Figure 4.7) was installed on each end of the tendon. Steel wedges were then inserted onto each strand. Each wedge was manually set into the anchor heads using a metal conduit. A steel post-tensioning button (see Figure 4.21(a)) was then installed onto the stressing end of the tendon. The button was equipped with automotive valve springs that helped restrain the wedges during post-tensioning.

(iii) Install hydraulic cylinder, pressure transducer, and stressing head:

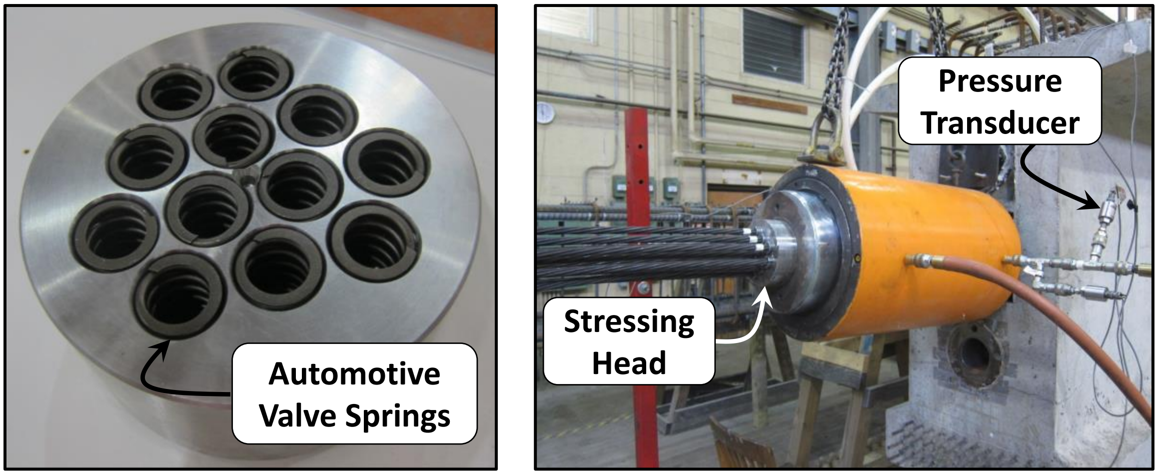
A hydraulic cylinder with a hollow plunger (i.e., center hole) was used to apply force to the post-tensioning strands, as shown in Figure 4.21(b). A pressure transducer, previously calibrated with the hydraulic cylinder, was installed in-line with the hydraulic system to monitor the applied force during the post-tensioning operations. After the hydraulic cylinder was positioned, the stressing head was installed onto the tendon, and steel wedges were set into the head using a metal conduit.

(iv) Post-tension the tendon:

After completing the preparation of the hydraulic system, stressing of the tendon began. Each tendon was post-tensioned in increments of approximately 20 percent of the final jacking stress to monitor elongation of the strands. The elongation was monitored by measuring the additional extension of the hydraulic cylinder during each load step and comparing the value to the expected elongation. Any slack in the strands was assumed to be removed at approximately 20 percent of the final jacking stress. The force in the tendon was continuously monitored using the pressure transducer, and a dial pressure gauge was available to verify the jacking force. Readings from vibrating wire gauges embedded in the concrete were also continuously recorded during the post-tensioning operation.

(v) Retract hydraulic cylinder and conduct final verification:

After the final jacking stress was reached, pressure was slowly released from the hydraulic cylinder. The stress in the tendon calculated using data from vibrating wire gauges was used to verify the final jacking stress, as described in Section 4.14.1. The stress in each tendon after the anchorage was set was determined using vibrating wire gauges and is provided in Section 4.17.



(a) *(b)*

Figure 4.21: Post-tensioning equipment – (a) post-tensioning button; (b) hydraulic cylinder used for post-tensioning

4.13.5 Grouting Procedure

Following the post-tensioning procedure, each tendon was grouted with thixotropic grout for prestressing strands. A grout plant, shown in Figure 4.22, was used to mix and pump the grout. A total of five grout vents were placed along the length of each tendon. Two vents were located at each end of the 50-ft long test girder. One of these vents extended from the top of the bearing trumplate while the other was located at the grout cap that was installed over the anchor head. The fifth vent was placed near the center of the girder near the low point of the tendon profile. One of the grout vents originating at the top of the bearing trumplate was typically used as the inlet for the grout to flow into the duct. Prior to the grouting procedure, the grout caps were installed at the ends of the girder and all vents were equipped with positive shut-off valves. Pressure gauges were located at the inlet to the duct and between the grout plant and the girder. The following procedure (based on the requirements in PTI M55.1-12) describes the steps performed for each grouting operation:

(i) Batch water:

Water was first measured by weight in accordance with the dosage recommended by the grout manufacturer (1.8 to 2.1 gallons per 55-lb bag of grout). The grout plant contained a water batching tank that was used to store water until it was needed.

(ii) Mix grout in colloidal mixer:

After adding water to the colloidal mixing tank, the mixer was started and grout was added to the tank (see Figure 4.23(a)). Once mixed, the grout was transferred to the agitator tank where a paddle mixer ensured the grout remained in motion.

(iii) Test wet density and flow of grout:

Both the wet density and flow of the grout mixture were tested to verify its quality and pumpability. The modified flow cone test was performed in accordance with Section 4.4.5.2 of PTI M55.1-12 (see Figure 4.23(b)) to ensure the efflux time was between 5 and 15 seconds (a 5 to 30 second range is recommended in PTI M55.1-12). A mud balance was then used per Section 4.4.8 of PTI M55.1-12 to verify that the wet density of the grout was greater than 1.95 g/cm^3 as recommended by the manufacturer (see Figure 4.23(c)). The flow and wet density of the grout was satisfactory for each grouting procedure conducted during the testing program.

(iv) Pump grout:

After the wet density and flow of the grout was verified, samples were taken to cast 2-in. cubes according to ASTM C1107 for future compression testing. The grout was then pumped into the duct. Each of the grout vents was closed in succession after approximately 2 gallons of grout were emitted from the outlet, as shown in Figure 4.23(d). Immediately after the last outlet was closed, the valve at the inlet was also closed, and the pump was then powered down.

Figure 4.22: Grout plant

Figure 4.23: Grouting procedure – (a) adding grout to colloidal mixing tank; (b) flow cone test; (c) measuring wet density with mud balance; (d) grout emitted from outlets

4.13.6 Deck Placement

The girders were moved to their final location for testing after the post-tensioning and grouting procedures were completed. The final step in the fabrication of the spliced girder test specimens was the placement of a deck over the length of the girders. The decks provided the girders with additional strength and caused them to be more representative of actual field members. Each deck had a thickness of 8 in. and a total width of 42 in. The width was 2 in. shorter than the top flange width to accommodate placement of the formwork on top of the girders. The composite girder section is shown in Figure 4.24. The concrete mixtures used for the decks are provided in Section 4.12, and the concrete compressive strengths at the time of testing are presented in Section 4.17.

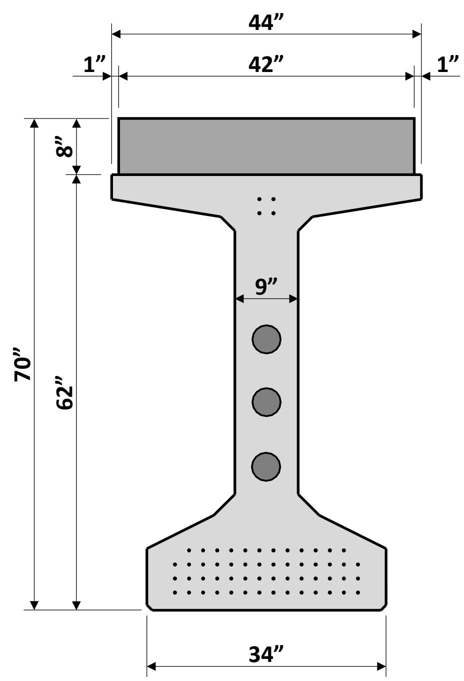


Figure 4.24: Test girder with deck dimensions

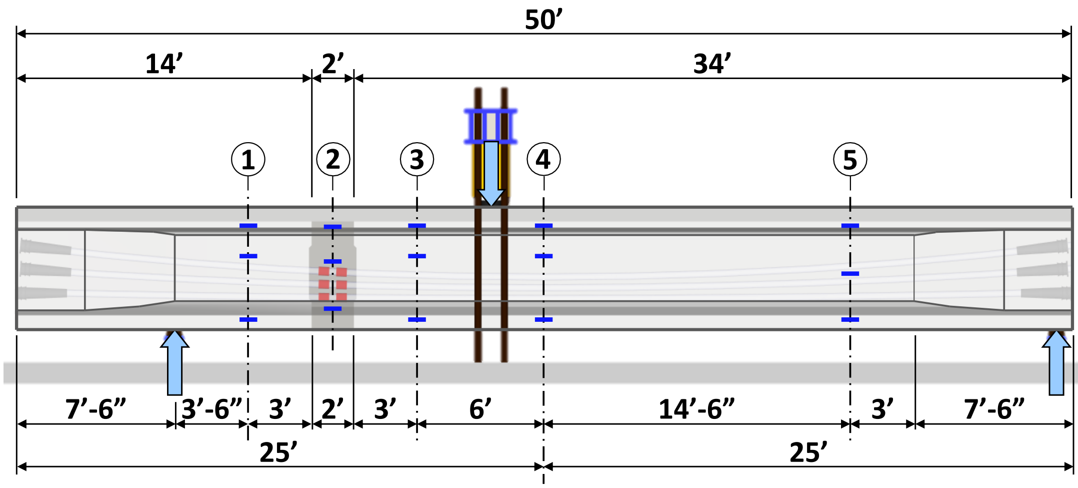
4.14 INSTRUMENTATION

Several sensor types were installed on the test girders or embedded in the concrete to aid in developing a more complete understanding of the behavior of the specimens. The manner in which sensors were used is detailed in this section. The sensor data most relevant to understanding the behavior of the test girders are described in Chapter 5.

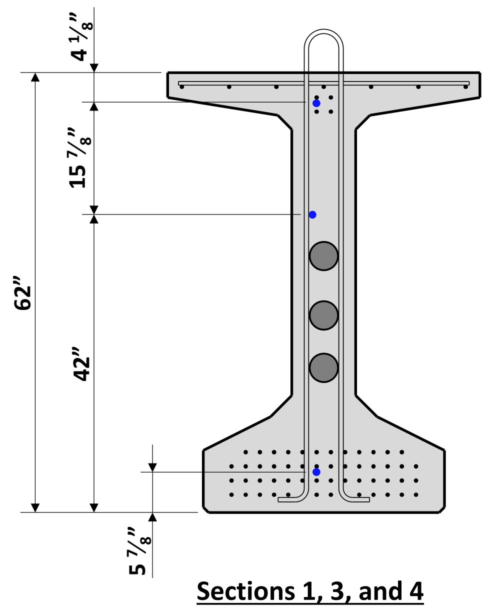
4.14.1 Vibrating Wire Gauges

Vibrating wire gauges (VWGs) were embedded in the precast segments and the splice region to measure concrete strains. A VWG attached to the longitudinal interface reinforcement within the web of the splice region of the first test girder is pictured in Figure 4.25. Each gauge was installed in the horizontal orientation as shown. As indicated in Figure 4.26(a), a total of 15 VWGs arranged at 5 different sections (labeled 1 through 5) were placed within each test girder. The approximate locations of the gauges within Sections 1, 3, and 4 are shown in Figure 4.26(b) as an example. After each VWG was installed, its exact location was measured and recorded to be used in post-processing computations.

Figure 4.25: Vibrating wire gauge within splice region



(a)



(b)

Figure 4.26: Vibrating wire gauge placement – (a) elevation view; (b) Sections 1, 3, and 4

As will be discussed in Section 5.2.2, the critical section of the test girders was located at the interface between the long precast segment and the splice region. Since the pretensioned strands were discontinuous at this location, the stress in the post-tensioning tendons governed the shear strength and was determined using data from the vibrating wire gauges. For each post-tensioning operation, the VWGs could be used to calculate the post-tensioning force at each of the 5 cross-sections shown in Figure 4.26(a) by applying the computational method illustrated in Figure 4.27. The change in strain of the concrete at the centroid of the cross-section of the girder, located at a distance $y_{transformed}$ from the bottom of the girder in Figure 4.27, was estimated from the strains indicated by the three VWGs placed within the section being considered. The post-tensioning force, P_{PT}, was then calculated using the equation provided in the figure, where $A_{transformed}$ is the transformed area of the cross-section and $E_{concrete}$ is the modulus of elasticity of the concrete (determined based on ASTM C469).

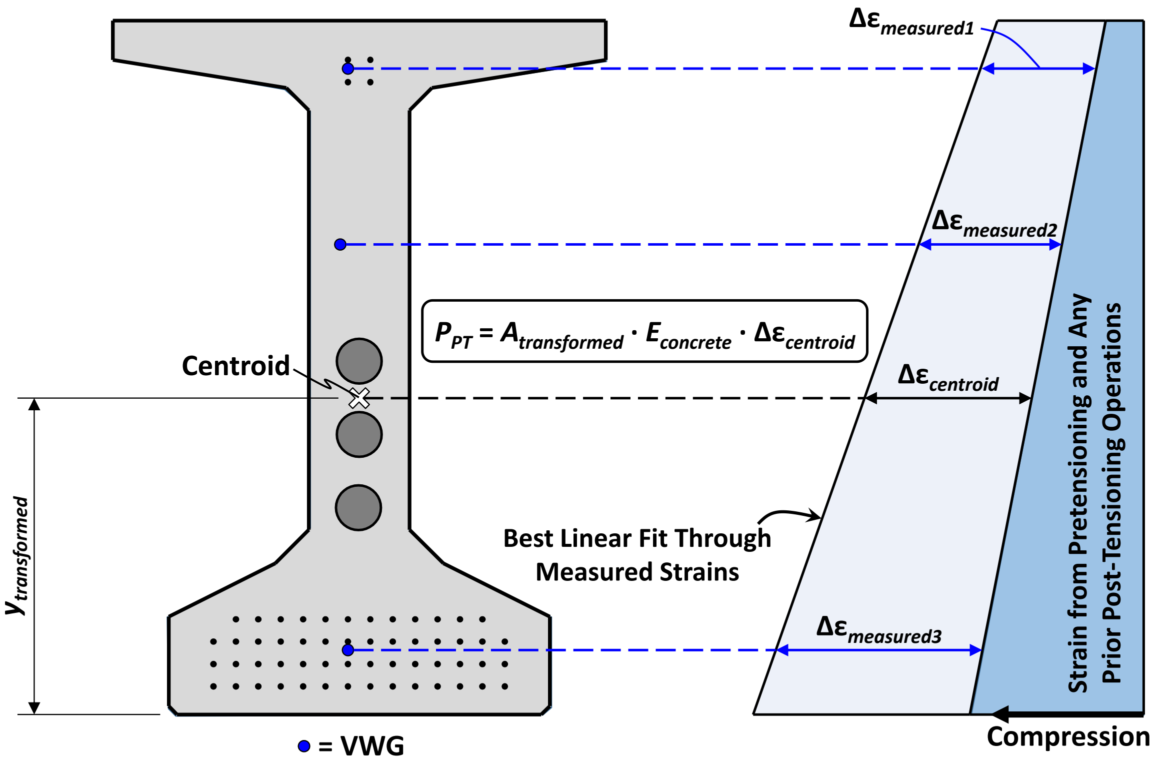


Figure 4.27: Calculating the post-tensioning force using vibrating wire gauges (adapted from Moore, 2014; based on Gallardo Méndez, 2014)

For each post-tensioning operation, the tendon force calculated from VWG data was compared to the force indicated by the pressure transducer to verify the final jacking stress. The VWG readings were then used to determine the losses experienced by each tendon when the anchorage was set. At the end of the post-tensioning operation for each tendon, the losses indicated by the VWGs at each of the 5 cross-sections were averaged. This value was subtracted from the final jacking stress indicated by the pressure transducer to obtain the stress in the tendon used in strength calculations. The VWGs did not present a clear trend of friction losses along the girder lengths. Therefore, any effect of these losses were neglected for comparisons of the calculated shear strengths of the girder specimens with their experimental capacities. The stress in each tendon of the two girder specimens at the end of each post-tensioning operation is provided in Section 4.17 below. Please see Section 5.4.4 for further discussion on post-tensioning losses.

The VWG data can also be used to determine additional post-tensioning losses (i.e., creep and shrinkage losses) prior to testing the specimens as well as losses in the pretensioned strands from the time of prestress transfer.

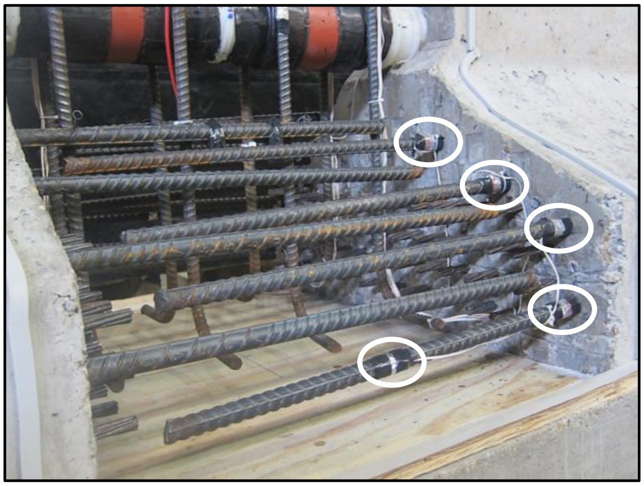
4.14.2 Foil Strain Gauges

Foil strain gauges were installed on the mild reinforcement in the splice regions of the girder specimens to monitor the change in strain of the steel during testing. They had a gauge length of 6 mm and a width of 2.6 mm. Their nominal resistance was 350 ohms (± 1.5 ohms).

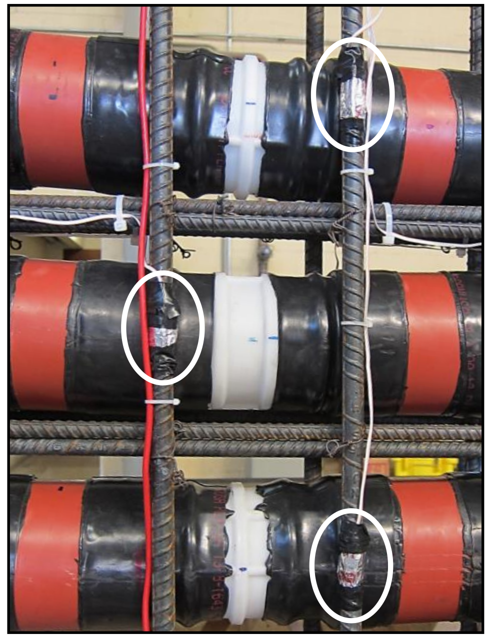
To install a foil gauge, a small area of the reinforcing bar was first lightly ground and polished to remove the bar deformations and create a smooth surface. Precautions were taken to ensure that very little cross-sectional area of the bar was removed. Adhesive was then used to affix the gauge to the rebar. Although the strain gauges were pre-coated with a waterproof epoxy resin, the gauges were covered with a combination of electrical and foil tape to provide further protection. The strain gauge installation procedure is demonstrated in Figure 4.28.

Figure 4.28: Strain gauge installation procedure

A total of 30 foil strain gauges were installed in the splice region of each girder specimen. Due to interest in the effect of the longitudinal interface reinforcement on the behavior of the spliced girders, a majority of the gauges were placed on the interface bars and centered at ½ in. from the faces of the precast girder segments. Strain gauges installed on interface bars extending from the long segment of the second girder specimen are pictured in Figure 4.29(a). Other gauges were affixed to the legs of the middle two stirrups within the splice regions. They were placed to coincide with the location of the post-tensioning ducts, as shown in Figure 4.29(b). Two additional gauges were also installed on the stirrups above the ducts in the second test girder.



(a)



(b)

Figure 4.29: Strain gauge placement in splice region – (a) longitudinal interface bars; (b) stirrup legs

The strain gauges were continuously monitored during each post-tensioning operation and during testing. The resulting data that is most relevant to understanding the role of the interface and shear reinforcement on the behavior of the test girders are detailed in Chapter 5. Plots for all the strain gauges are provided in Appendix F.

4.14.3 Linear Potentiometers

The test specimens were instrumented with linear potentiometers to measure the vertical deflection of the girder as well as any relative displacements and flexural deformations at the splice region. The placement of linear potentiometers at the splice region is presented in Figure 4.30. Aluminum mounting brackets and plates were installed on the girder web to facilitate measurement of the relative vertical displacements between the precast segments and the splice region in addition to the opening of any vertical cracks at the splice region interface (refer to parts (a) through (c) of Figure 4.31).

Three linear potentiometers were also attached to the bottom surface of the girder, as shown in Figure 4.31(d), to monitor the opening of flexural cracks at the splice region interface or within the splice region itself. Epoxy was used to affix all mounting brackets and plates to the concrete surface. Additional linear potentiometers were placed on floor stands to capture vertical displacements within and immediately outside the splice region, as pictured in Figure 4.31(e), as well as the deflections at the load point and support bearings (see Figure 4.32). Data from all the linear potentiometers are presented in Appendix F.

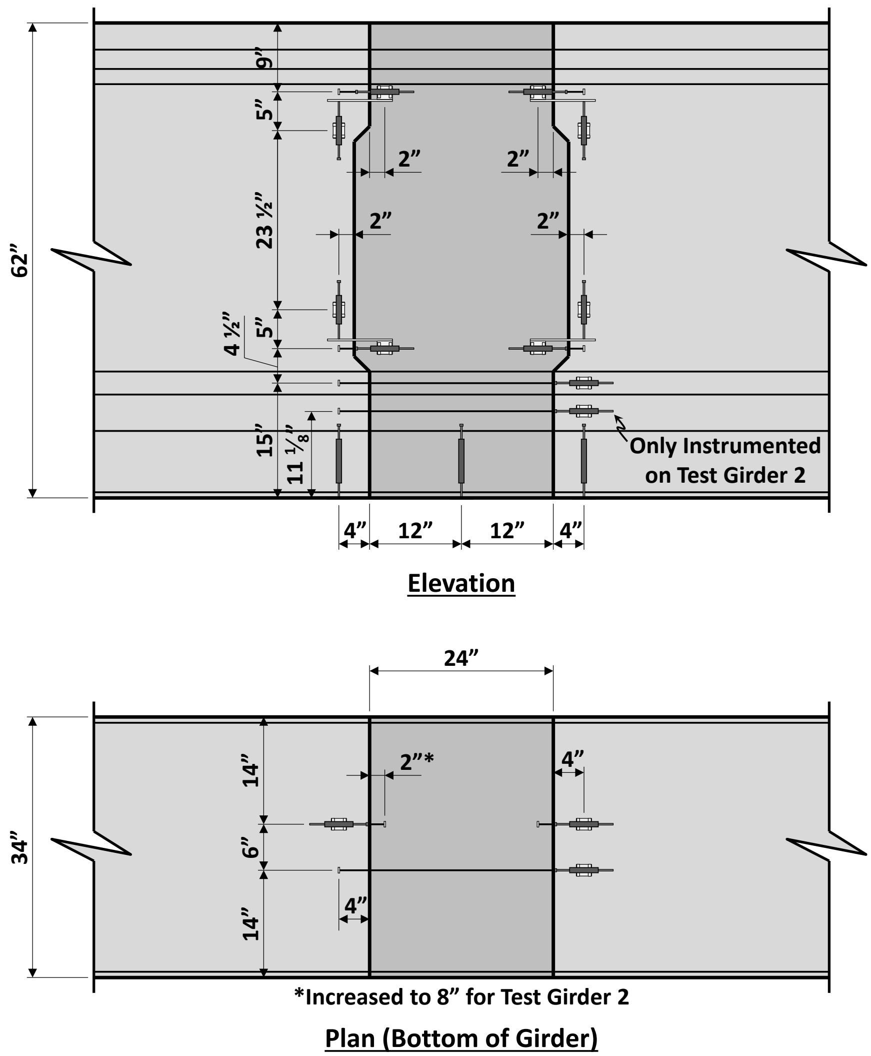
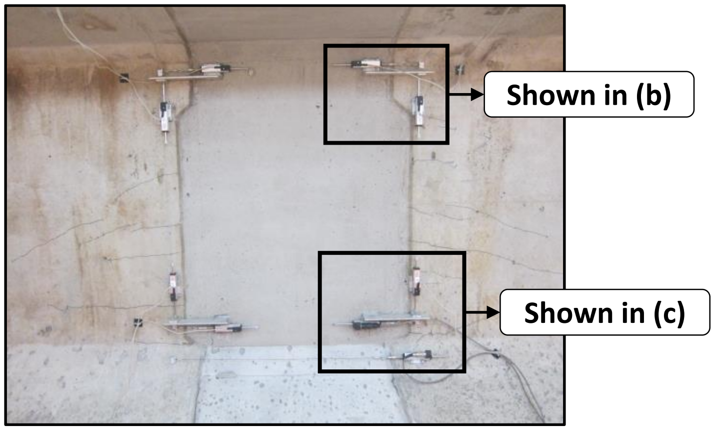
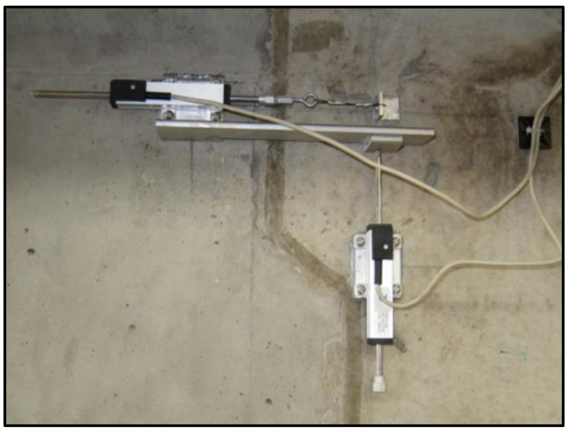


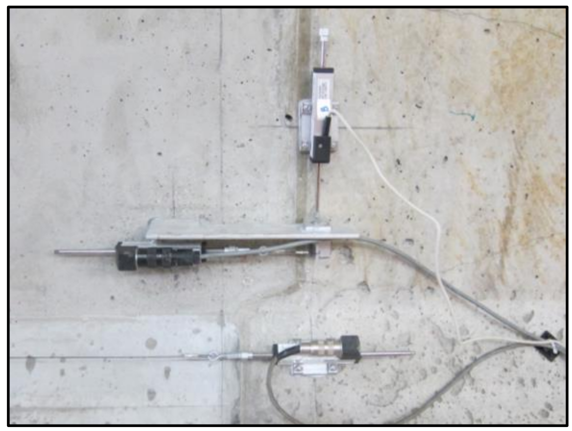
Figure 4.30: Linear potentiometer placement at splice region



(a)



(b)



(c)

(d)

(e)

Figure 4.31: Linear potentiometer instrumentation – (a) girder web; (b) near top of web; (c) near bottom of web; (d) bottom surface of girder; (e) measuring vertical displacement

4.14.4 Load Cells and Pressure Transducer

The reactions at each support location were measured with two 1000-kip capacity load cells, as shown in Figure 4.32. For each test specimen, load readings were captured at the time the girder was placed on the supports to determine its self-weight and the corresponding shear within the test span. Measurements were also taken after the load frame was installed to obtain the dead load shear it imposed on the girder. These values were added to the applied shear measured during testing to calculate the total shear force within the test span. To ensure accuracy, a pressure transducer was attached to the hydraulic cylinder to verify the applied load indicated by the four load cells, as noted in Figure 4.32.

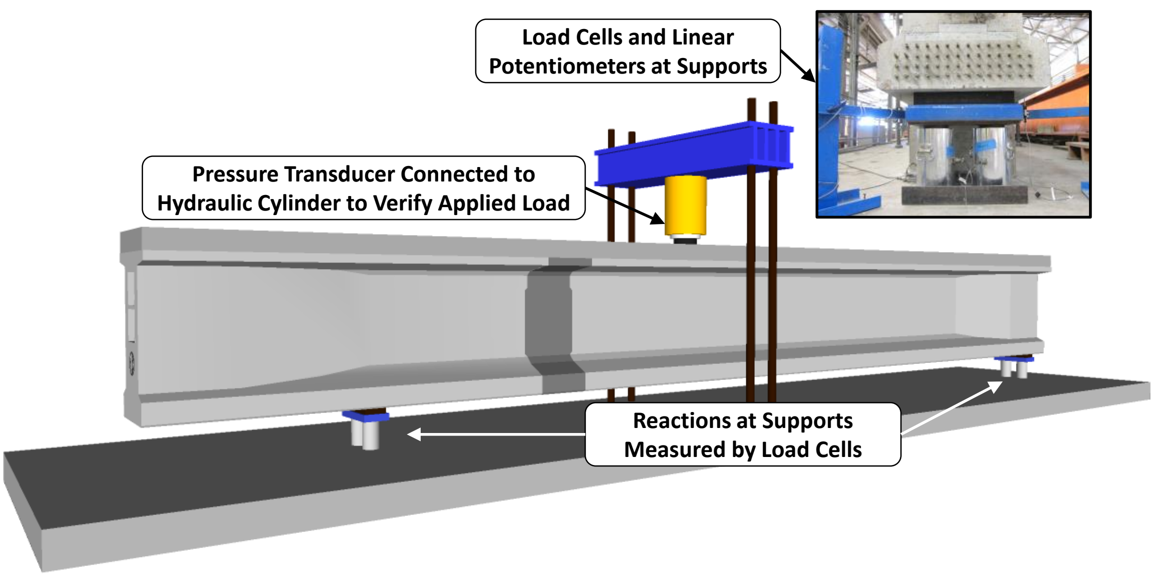


Figure 4.32: Load cell and pressure transducer instrumentation (adapted from Moore, 2014)

4.15 LOADING CONFIGURATION

The loading configuration for the spliced girder testing program is illustrated in Figure 4.33. The girders were simply supported, and a 2,000-kip capacity load frame was installed to apply load to the specimens. The test span was 15 ft in length with the 2-ft long splice region centered within the span. To eliminate any effects of the thickened web on the behavior of the girder, the end block was located outside the test span by allowing it to overhang the support (i.e., the support was placed 7 ft-6 in. from the girder end). The far support was located 9 in. from the opposite end of the test girder, giving a back span length of 26 ft-9 in. The test setup was configured to ensure the splice region would experience both high shear and flexural demands as the ultimate load was approached, creating a critical loading scenario. At the same time, the configuration was designed to cause the girders to exhibit a shear failure. This failure mode was selected as the most critical failure mode that is likely to be influenced by the presence of post-tensioning ducts and duct couplers.

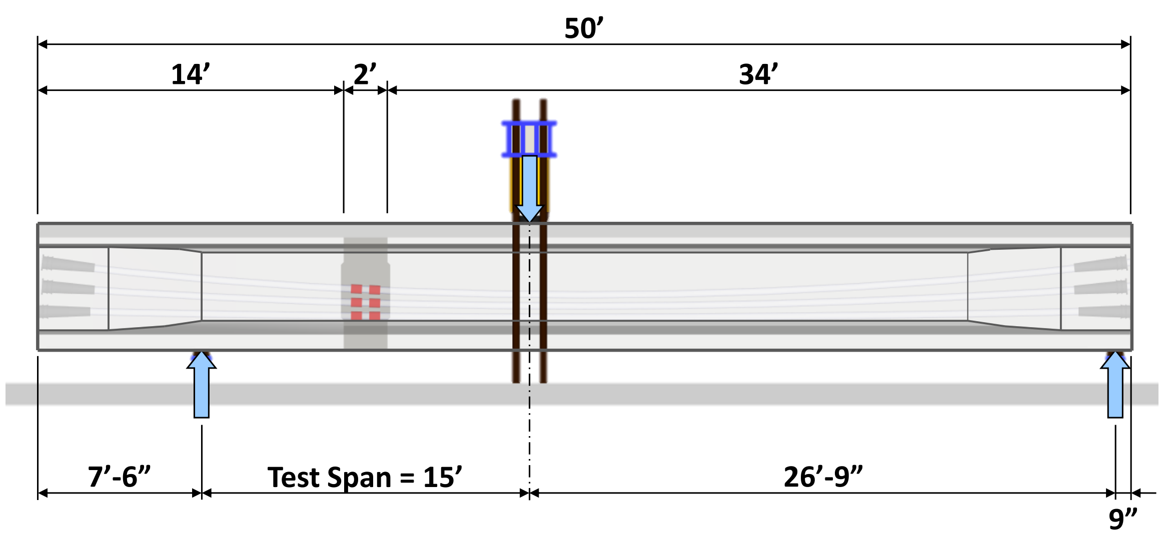


Figure 4.33: Loading configuration

4.16 TEST PROCEDURE

Each of the two test girders were loaded monotonically until the specimen exhibited a shear-compression failure of the web concrete. Load was applied in increments of 100 kips or less. After each load increment up to an applied load of 800 kips, the girders were visually inspected to detect the formation or growth of cracks, and any significant observations were recorded. New cracks or crack extensions were marked with felt-tipped markers and labeled with the corresponding applied load (see Figure 4.34(a)). Furthermore, the widths of both shear and flexural cracks were monitored throughout the tests using a crack comparator card (see Figure 4.34(b)). For safety concerns as the specimens approached failure, cracks were not marked or measured after an applied load of 800 kips was reached. Photographs were taken throughout the testing procedure, and the failure of each test girder was video recorded.

(a) *(b)*

Figure 4.34: Marking and measuring cracks during testing – (a) marking cracks with felt-tipped markers; (b) measuring crack widths

4.17 QUANTITATIVE TEST SPECIMEN DETAILS

A summary of the quantitative test specimen details for the two spliced girders is presented in Tables 4.5 through 4.7. The variables used in the tables are defined as follows:

A_{ps} = Area of prestressing steel (in.^{2})

f'_c = Compressive strength of concrete or grout (ksi)

f'_t = Splitting tensile strength of concrete at the time of testing (ksi)

f_{vy} = Measured yield strength of vertical shear reinforcement (ksi)

Stress = Stress in post-tensioning tendon at the end of the post-tensioning operation, calculated as described in Section 4.14.1 (ksi)

y_p = Distance from the bottom of the girder to the centroid of the post-tensioning tendon at the critical section (in.)

ρ_v = Area of shear reinforcement divided by the gross concrete area of a section taken on a horizontal plane

The compressive strength of concrete, f'_c, corresponding to the time of post-tensioning the tendons is presented in Table 4.5. The number of days since the concrete was cast is also provided. For the precast concrete, one strength value is given for each girder due to the maturity of the concrete when the specimens were post-tensioned.

Table 4.5: Compressive Strength of Concrete at Time of Post-Tensioning

	Splice Region Concrete							
	Top Tendon (Stressed First)		Middle Tendon (Stressed Second)		Bottom Tendon (Stressed Third)		Precast Concrete	
Test Specimen	Age	f'_c (ksi)	Age	f'_c (ksi)	Age	f'_c (ksi)	Age	f'_c (ksi)
Girder 1	13 Days	7.03	22 Days	7.65	30 Days	8.43	>140 Days	13.63
Girder 2	7 Days	7.20	16 Days	8.28	25 Days	9.08	>180 Days	13.81

Measured properties of the precast, splice region, and deck concretes corresponding to the time of testing the girder specimens are provided in Table 4.6. The reinforcement ratio, ρ_v, and yield strength, f_{vy}, of the transverse shear reinforcement is also presented.

Table 4.6: Summary of Material Properties (Corresponding to the Time of Testing)

Test Specimen	Transverse Reinforcement		Precast Segments		Splice Region		Deck
	ρ_v (%)	f_{vy} (ksi)	f'_c (ksi)	f'_t (ksi)	f'_c (ksi)	f'_t (ksi)	f'_c (ksi)
Girder 1	1.15	62.0	13.88	0.98	9.48	0.90	6.50
Girder 2	1.15	67.7	14.54	0.97	10.07	0.87	9.72

Details related to the post-tensioning tendons are summarized in Table 4.7. The tendon locations correspond to the critical section described in Section 5.2.2.

Table 4.7: Summary of Post-Tensioning Tendon Details

Test Specimen	Tendon	Grout	Post-Tensioning Strand			
		f'_c (ksi)*	Force (kip)	Stress (ksi)	A_{ps} (in.^{2})	y_p** (in.)
Girder 1	Bottom (1)	8.71	492	189	2.604	20.1
	Middle (2)	9.40	495	190	2.604	26.9
	Top (3)	8.76	471	181	2.604	33.8
Girder 2	Bottom (1)	10.94	487	187	2.604	20.1
	Middle (2)	10.87	479	184	2.604	26.9
	Top (3)	10.59	482	185	2.604	33.8

* At the time of testing the girder specimens

**A 1-in. offset of the tendon from the center of the duct is assumed

4.18 SUMMARY

The details of the experimental program conducted to study the behavior of spliced I-girders were described in this chapter. Two girder specimens were tested, each consisting of two precast segments joined at a cast-in-place splice region. Continuity was provided by three post-tensioning tendons that extended the full 50-ft length of each girder. The structural performance of the details within the splice regions were of primary interest, and the reasons for selecting each detail were discussed. The amount of longitudinal interface reinforcement within the bottom flanges of the test girders was varied between the two specimens to study the effect of the bars on the behavior of the splice regions.

Several steps were required to fabricate the precast segments and conduct the splicing operations. The girders were tested in a 2,000-kip capacity load frame until the specimens failed in shear. Several instruments, such as strain gauges, linear potentiometers, and load cells, were used to monitor the behavior of the girders during testing.

The analysis of experimental results and observations from the spliced girder testing program is described in Chapter 5. The shear-friction experimental program conducted to supplement the splice region research is introduced in Chapter 6.

Chapter 5. Analysis of Experimental Results and Observations

5.1 Introduction

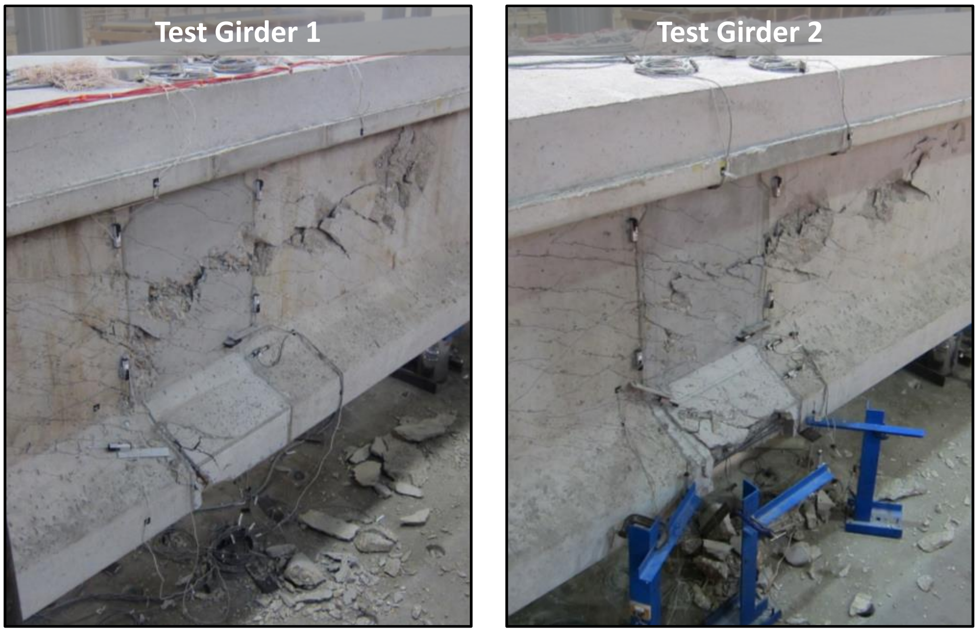
The results of the load tests performed on the two spliced girder specimens are discussed in detail in this chapter. A basic overview of the test results is followed by a more in-depth description of the strength and serviceability behavior of the test girders. Primary emphasis is placed on the behavior of the cast-in-place (CIP) splice regions. Data from the various sensor types monitored during the load tests are presented and analyzed to gain a better understanding of the behavior of spliced girders. Furthermore, the effect of the longitudinal interface reinforcement extending from the precast segments into the splice regions is highlighted.

5.2 Overview of Experimental Results

The results of the spliced girder testing program provide significant insights into the strength and structural behavior of CIP splice regions. To provide an overview of the test results, the following section describes the observed failure mechanism of the girders. Load-deflection plots are also presented following an explanation of how the shear force acting at the critical section was determined.

5.2.1 Shear-Compression Failure Mechanism

In an effort to study the structural performance of cast-in-place splice regions, the test specimens were designed to exhibit a shear failure. This failure mode was selected as the most critical failure mode that is likely to be influenced by the presence of post-tensioning ducts and duct couplers. Consistent with their design and observations from previous shear tests on post-tensioned girders, the failure mechanisms of both specimens were defined by a shear-compression failure within the thin webs. The primary concrete crushing was located in the vicinity of the top post-tensioning duct. The failure mechanism was consistent with the behavior of the monolithic post-tensioned specimens of the spliced girder research program which contained a single duct located at the mid-height of the girder webs. Photographs of the two spliced girder test specimens after failure are shown in Figure 5.1.



(a) *(b)*

Figure 5.1: Test girders after failure – (a) Test Girder 1; (b) Test Girder 2

The crack patterns after failure of the girders are provided in Figures 5.2 and 5.3. The two illustrations reveal that a majority of the concrete crushing was located near the top duct. The cracks shown in green in Figures 5.2 and 5.3, as well as in other crack maps in this chapter, were preexisting before the start of the load tests (refer to Section 5.3). The spalling of concrete from the bottom flange within the splice region at later stages of loading is discussed in Section 5.5.

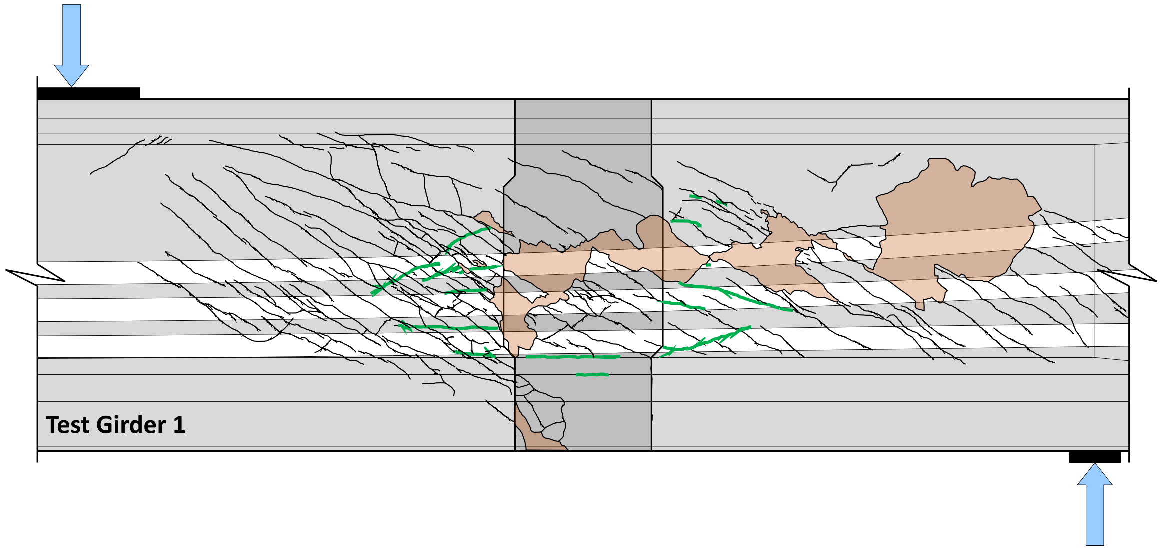


Figure 5.2: Crack pattern after failure – Test Girder 1

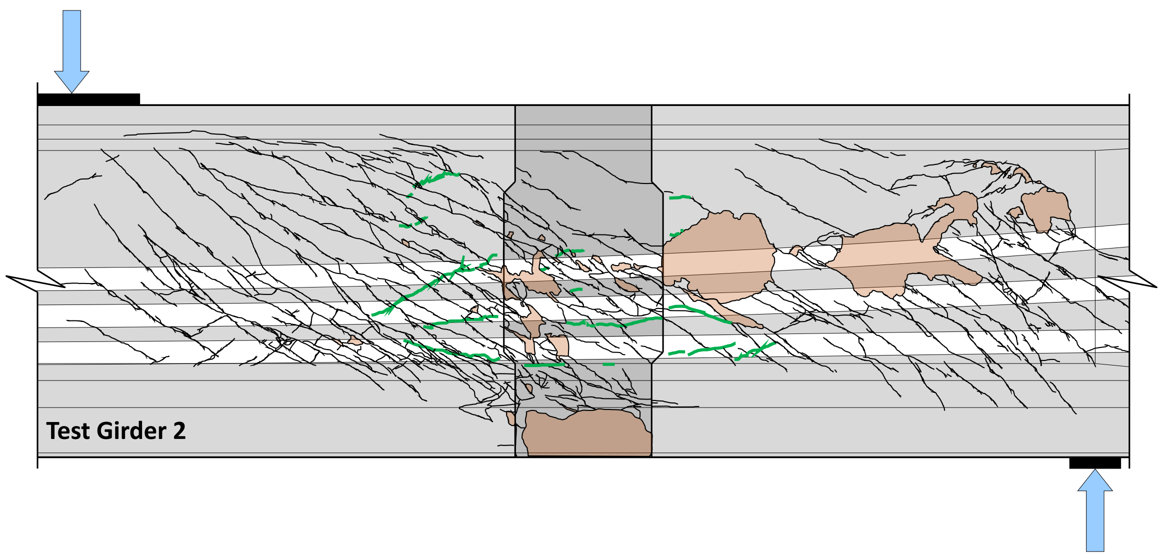


Figure 5.3: Crack pattern after failure – Test Girder 2

Based on the observed behavior of the specimens as indicated in Figures 5.1 through 5.3, the girders acted essentially as monolithic members in shear at failure. Web crushing extended across much of the test spans and was not localized within the splice regions. This seemingly simple observation is viewed to be the most significant experimental observation in view of the primary objectives of this research program.

5.2.2 Critical Section and Calculation of Shear Force

For purposes of evaluating the strength of the test girders, the critical section was taken as the location at the splice region with the lowest calculated shear strength, considering the given loading conditions, according to the general shear procedure of Article 5.8.3.4.2 of AASHTO LRFD (2014). The critical section was determined to be located at the interface between the long precast segment and the splice region, as indicated in Figure 5.4.

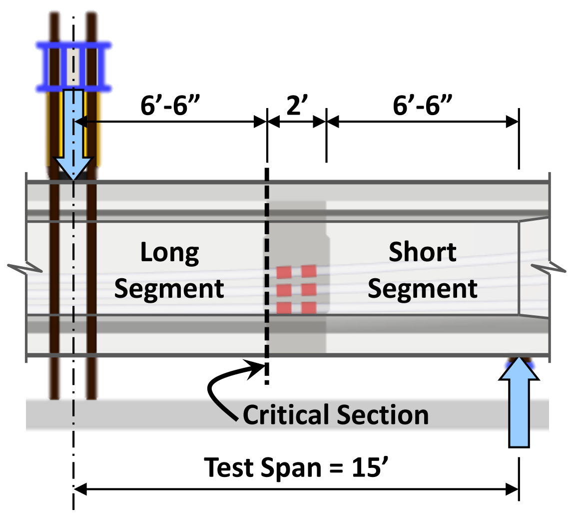
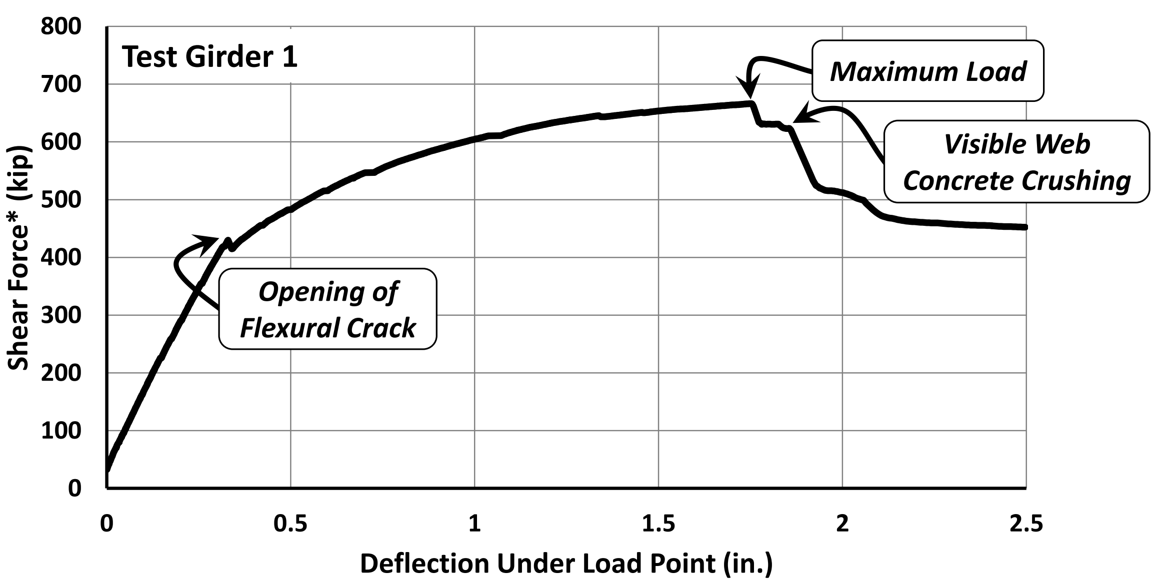


Figure 5.4: Location of critical section

The shear force acting at the critical section of the test specimens consisted of the effects from the self-weight of the girders as well as the force applied at the location of the load frame. The ultimate shear force, V_{test}, was therefore calculated by summing the shear force at the critical section due to the self-weight of the girder, the shear from the weight of the load frame itself, and the maximum shear applied to the test region by the hydraulic cylinder. The weight of each girder as well as the shear force applied to the test region was measured by load cells that were located at the supports, as described in Section 4.14.4. For consistency, much of the data presented in the current chapter are plotted versus the shear force acting at the critical section, including the effects of the girder self-weight and the weight of the load frame.

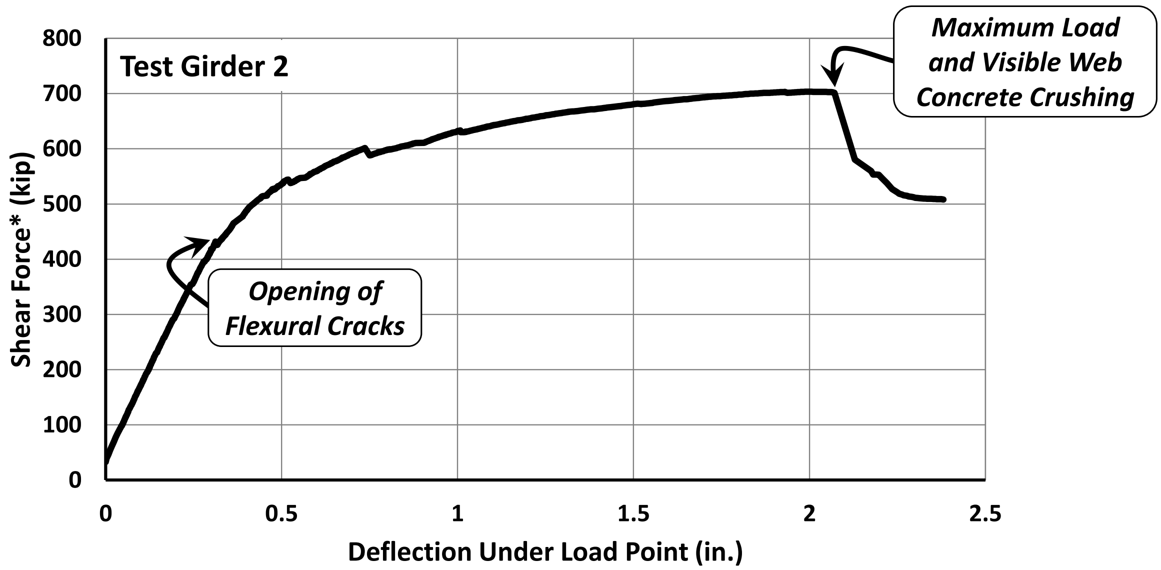
5.2.3 Load-Deflection Behavior

The overall behavior of the test girders can be described by the load-deflection plots provided in Figures 5.5 and 5.6. The shear force at the critical section indicated in Figure 5.4 is plotted versus the deflection measured under the load point. Notable events during the loading of the test girders are labeled on the plots. The deflection measurements were obtained by averaging the output from two linear potentiometers located on opposite sides of the girders at the location of the load point. Rigid body motion indicated by displacements measured at the supports was subtracted from the deflection values.



*Includes shear force from self-weight of girder and load frame (Refer to Section 5.2.2)

Figure 5.5: Load-deflection plot of Test Girder 1



*Includes shear force from self-weight of girder and load frame (Refer to Section 5.2.2)

Figure 5.6: Load-deflection plot of Test Girder 2

Considering the load-deflection plots, the behavior of the girders was fairly linear until flexural cracking at the splice region resulted in a notable reduction in stiffness. Upon further loading, the stiffness continued to decrease until the maximum load was reached. The occurrence of shear failure was accompanied by a significant reduction in load-carrying capacity and indicated by crushing of the web concrete in the vicinity of the top post-tensioning duct.

5.3 Evaluation of Service-Level Shear Behavior

The behavior of the spliced girder specimens under service-level shear loads was evaluated during the experimental program. To estimate the service-level loads for the test girders, the procedure introduced in Birrcher et al. (2009) and outlined in Figure 5.7 was followed. This technique was also used in Moore (2014) to estimate the service-level shear forces for the tests of the first phase of the spliced girder research program. In Figure 5.7, the LRFD strength equation was first rearranged to define the ratio of the resistance factor, ϕ, to the load factor, η, in terms of the service-level load and calculated nominal resistance. The value of the load factor was estimated to be 1.4 based on the load case assumed to govern the design (1.25*DL* + 1.75*LL*) as well as the assumed relationship between the dead and live loads. Performing the calculation presented in Figure 5.7, the service-level shear force was estimated to be approximately equal to $0.6V_{test}$. For this calculation, the ratio of the experimental shear capacity, V_{test}, to the nominal shear resistance, V_n, was taken as the average of the V_{test}/V_n values that resulted from the application of the AASHTO LRFD (2014) general shear procedure in evaluating the two spliced girder specimens (refer to Table 5.1 in Section 5.4.4). Furthermore, the AASHTO LRFD resistance factor for shear (0.90) was used. It should be noted that inputting the average V_{test}/V_n value resulting from the proposed shear design procedure (Table 5.1) would have provided a different estimate for the service-level shear force.

$$\phi * \text{Nominal Resistance} \approx \eta * \text{Service–Level Load}$$

$$\frac{\phi}{\eta} \approx \frac{\text{Service–Level Load}}{\text{Nominal Resistance}}$$

Assumptions:

$$\left.\begin{array}{l} \text{1) Load Case: } 1.25DL + 1.75LL \\ \text{2) } DL = 75\% \text{ of Service Load} \\ \quad\; LL = 25\% \text{ of Service Load} \end{array}\right\} \eta = 1.4$$

3) $V_{test}/V_n = 1.06$ (Table 5.1)

$$\frac{V_n}{V_{test}} * \frac{\phi}{\eta} = \frac{1}{1.06} * \frac{0.90}{1.4} = 0.6 \approx \frac{\text{Service–Level Load}}{\text{Experimental Capacity}}$$

Notation:

DL	=	Dead load	η	=	Load factor
LL	=	Live load	ϕ	=	Resistance factor, 0.90

Figure 5.7: Estimation of service-level loads as a function of the experimental capacity (Birrcher et al., 2009; Moore, 2014)

The service-level shear force estimate of $0.6V_{test}$ was determined only after making several assumptions. Adjustments to any of these assumptions would result in a different value. The estimate of $0.6V_{test}$ is therefore only a rough approximation for the evaluation of the spliced girder test specimens and is not meant to be a limit used in design. Due to the approximate nature of the service-level shear force estimate and in an attempt to include all relevant observations during the tests, the assessment of the service-level behavior within this section includes shear cracking that was noted at shear forces between 28 and 73 percent of the maximum shear force, V_{test}. A similar approach was followed in Moore (2014). Designers may choose to limit service loads on a spliced girder based on the predicted diagonal shear cracking behavior of the member. Moreover, the desire to prevent flexural cracking under service loads may govern the service-level design. Additional discussions regarding the flexural behavior as well as displacements at the splice region are presented in Section 5.4.

Prior to examining the behavior of the girders during the load tests, it is important to note the existence of cracks within the end regions of the precast girder segments that formed due to the transfer of the pre-tensioning force to the concrete. As an example, the cracks in the non-thickened end region (i.e., the end region to be spliced) of the short precast segment of the first test girder are shown in Figure 5.8. As described in Section 4.6, supplementary vertical reinforcement was placed within the non-thickened end regions to provide splitting resistance. Additional reinforcement may have aided in controlling the end-region cracks.

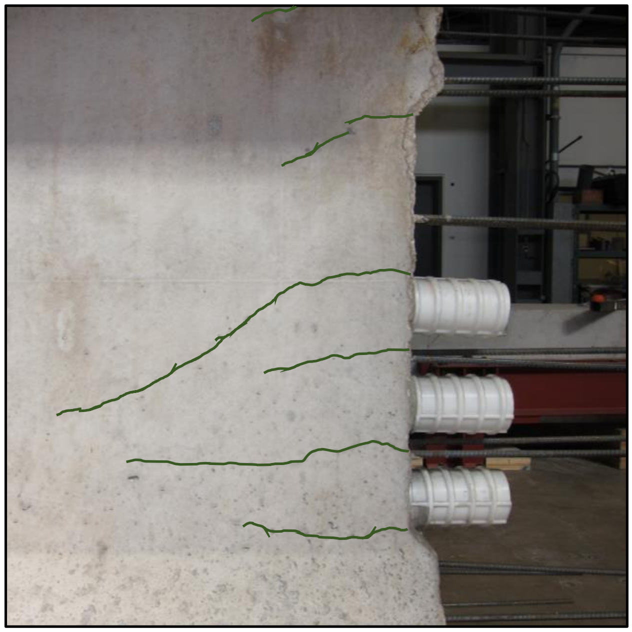
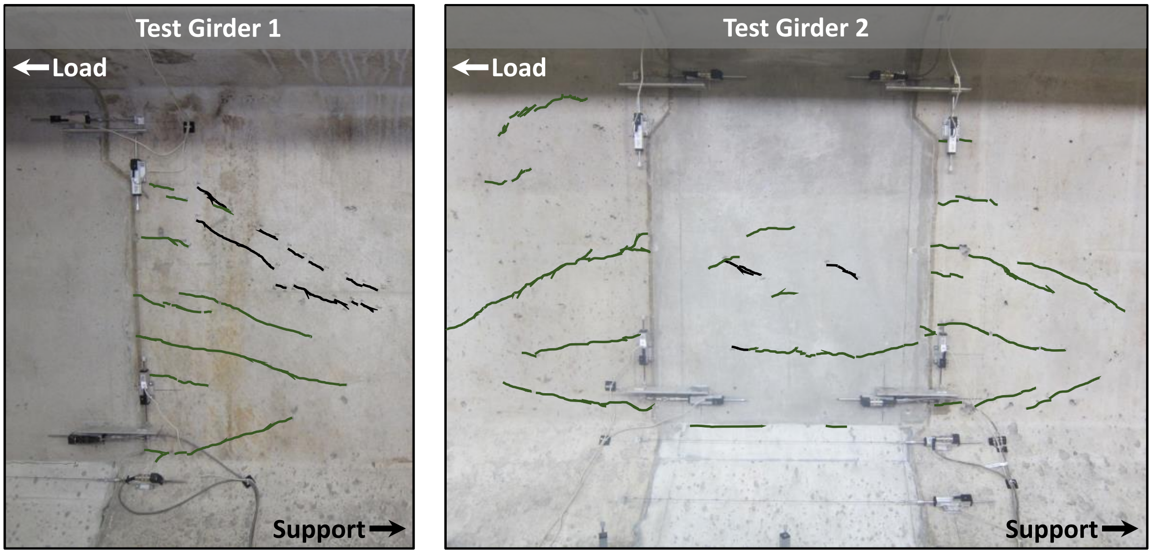


Figure 5.8: End-region cracking – short precast segment of Test Girder 1

The first cracks that formed during the load tests of the spliced girders are presented in Figure 5.9. Both specimens exhibited the formation of localized diagonal hairline cracks. For Test Girder 1, the cracks were first noted at a total shear force of 226 kips, 34 percent of the maximum shear force, V_{test}. The cracks may have developed, however, at a somewhat lower load since the formation of cracks was not checked between a shear force of 162 kips and 226 kips. As shown in Figure 5.9(a), the cracks formed outside of the splice region, possibly due to existing strains in the concrete caused by the pre-tensioning force. The development of cracks during the load test of Test Girder 2 were first observed at a total shear force of 194 kips, 28 percent of V_{test}. Unlike the first test girder, however, the cracks were located within the splice region, primarily in the vicinity of the top post-tensioning duct, and may have been influenced by the location of the relatively large plastic duct couplers. It should be noted that they were consistent with the hairline cracks that formed along the post-tensioning ducts of the monolithic test girders (Moore, 2014). The cracks marked green within the splice region of Test Girder 2 in Figure 5.9(b) were likely caused by shrinkage of the cast-in-place concrete and existed prior to the start of the load test.

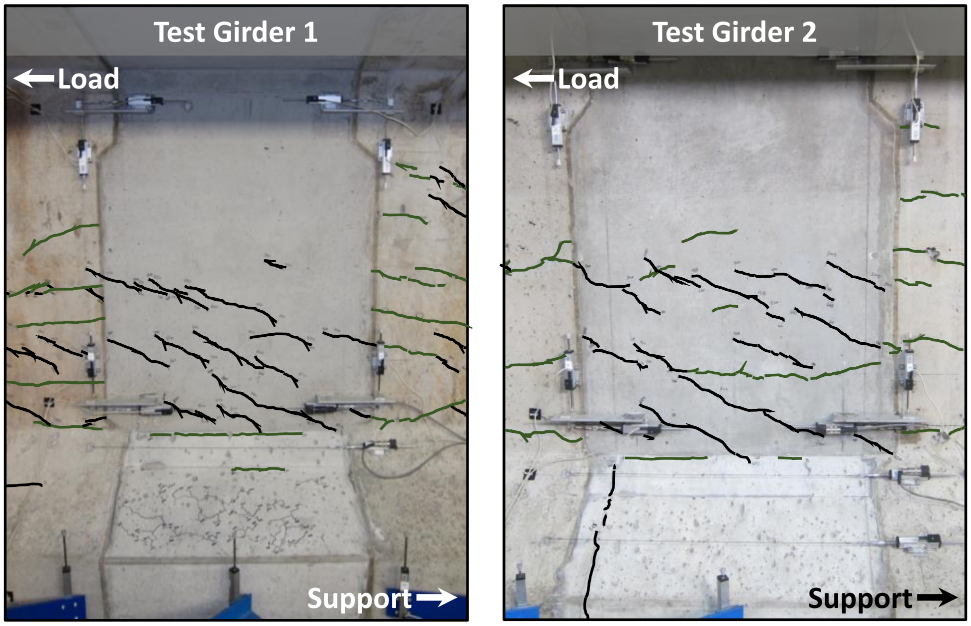


V = 226 kips (= $0.34V_{test}$)
(a)

V = 194 kips (= $0.28V_{test}$)
(b)

Figure 5.9: First cracks during load tests – (a) Test Girder 1; (b) Test Girder 2

Upon further loading of the girder specimens, localized cracks continued to extend and form in the test region. The cracks tended to develop in the vicinities of the three post-tensioning ducts, as shown in Figure 5.10 at a total shear force of approximately 420 kips (63 percent and 59 percent of V_{test} for the first and second test girders, respectively).

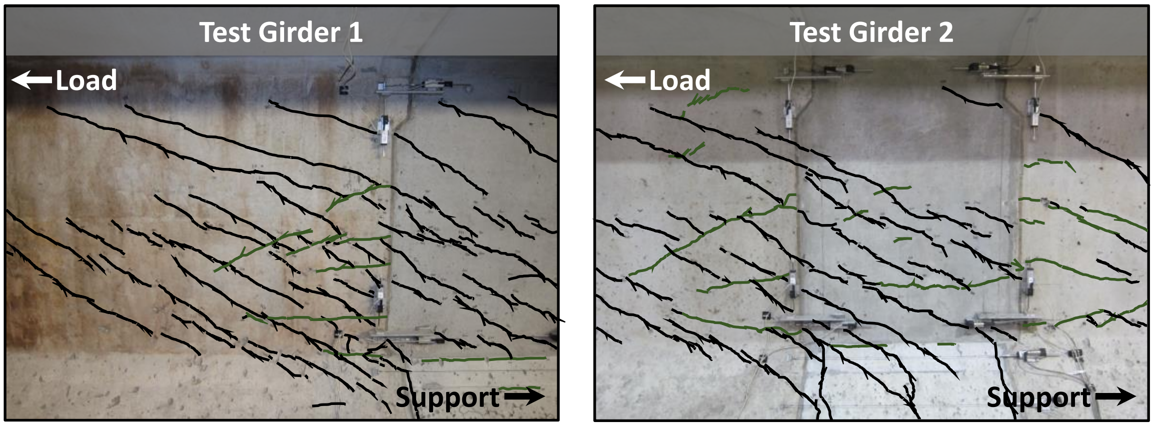


V = 419 kips (= 0.63V_{test})
(a)

V = 418 kips (= 0.59V_{test})
(b)

Figure 5.10: Distribution of cracks within the splice region – (a) Test Girder 1; (b) Test Girder 2

Both test girders exhibited shear cracks that extended over much of the web depth at a shear force of 483 kips, as presented in Figure 5.11. This cracking behavior was consistent with the first cracks observed during shear tests of pretensioned I-girders without post-tensioning ducts (Avendaño and Bayrak, 2008). The behavior was also comparable to the formation of shear cracks within the webs of the post-tensioned monolithic girder specimens of the spliced girder research program (Moore, 2014).



V = 483 kips (= 0.73V_{test})
(a)

V = 483 kips (= 0.69V_{test})
(b)

Figure 5.11: Cracks extending through the web – (a) Test Girder 1; (b) Test Girder 2

The shear cracking behaviors of the two test girders are summarized in Figures 5.12 and 5.13. The crack patterns and measured crack widths correspond to the west side of the girder specimens. The shear force, V, acting at the critical section is provided for each load step represented in the figures along with the ratio V/V_{test}. Any cracks that existed prior to the start of the tests are again marked in green. The width, w, of the shear crack measured at the location indicated by the red circle on the individual crack maps is also presented. For the first girder specimen, the width of one of the first shear cracks to develop during the test was monitored at each load step. During the test of the second girder specimen, an attempt was made to locate and measure the shear crack with the largest width at each load step. Comparing the two girders, the measured crack widths at similar shear forces were the same. It should be noted that only the widths of shear cracks that formed during the tests were considered, not the widths of preexisting cracks (i.e., shrinkage cracks or cracks that formed due to transfer of the pre-tensioning force to the concrete).

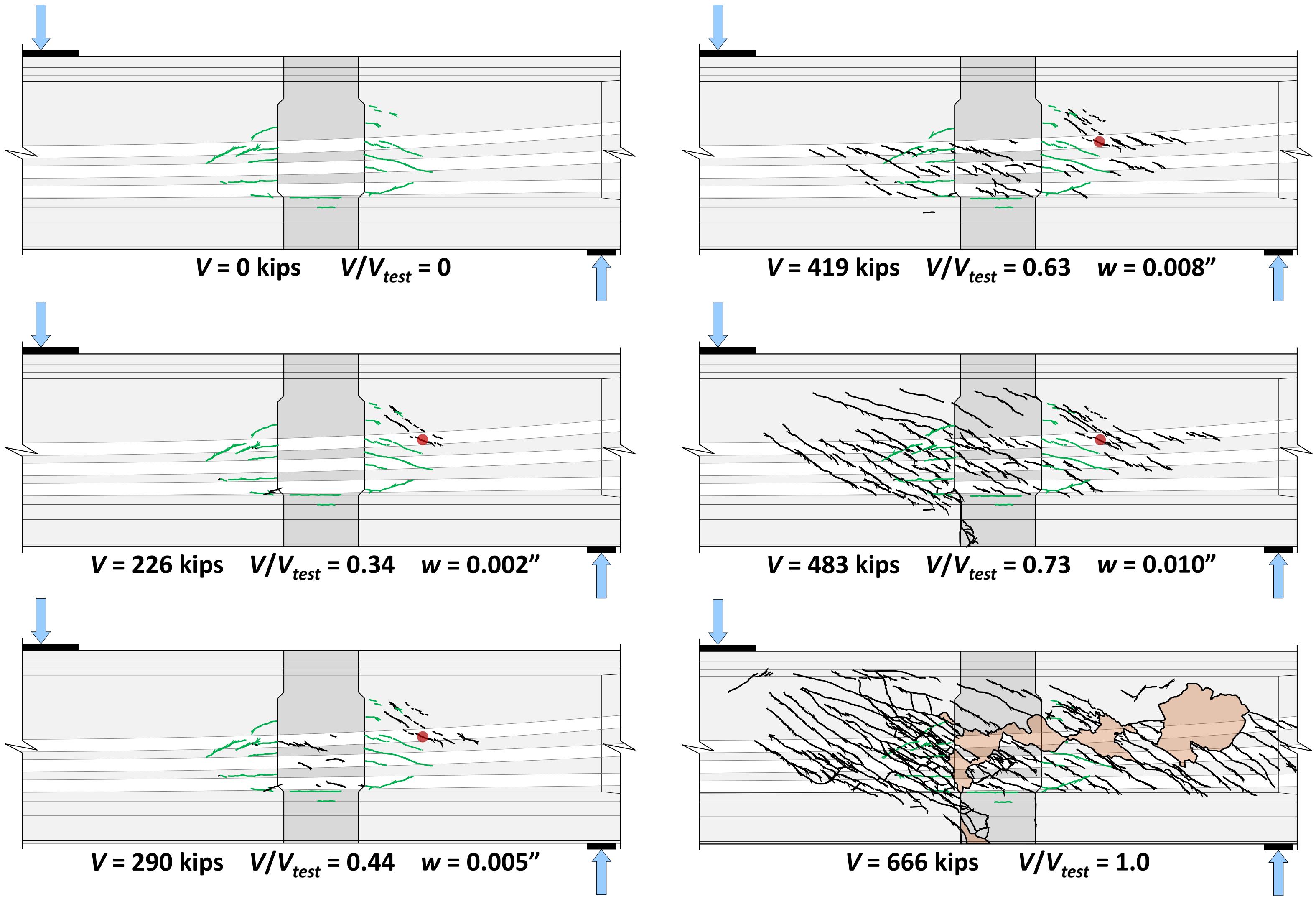


Figure 5.12: Summary of cracking behavior - Test Girder 1

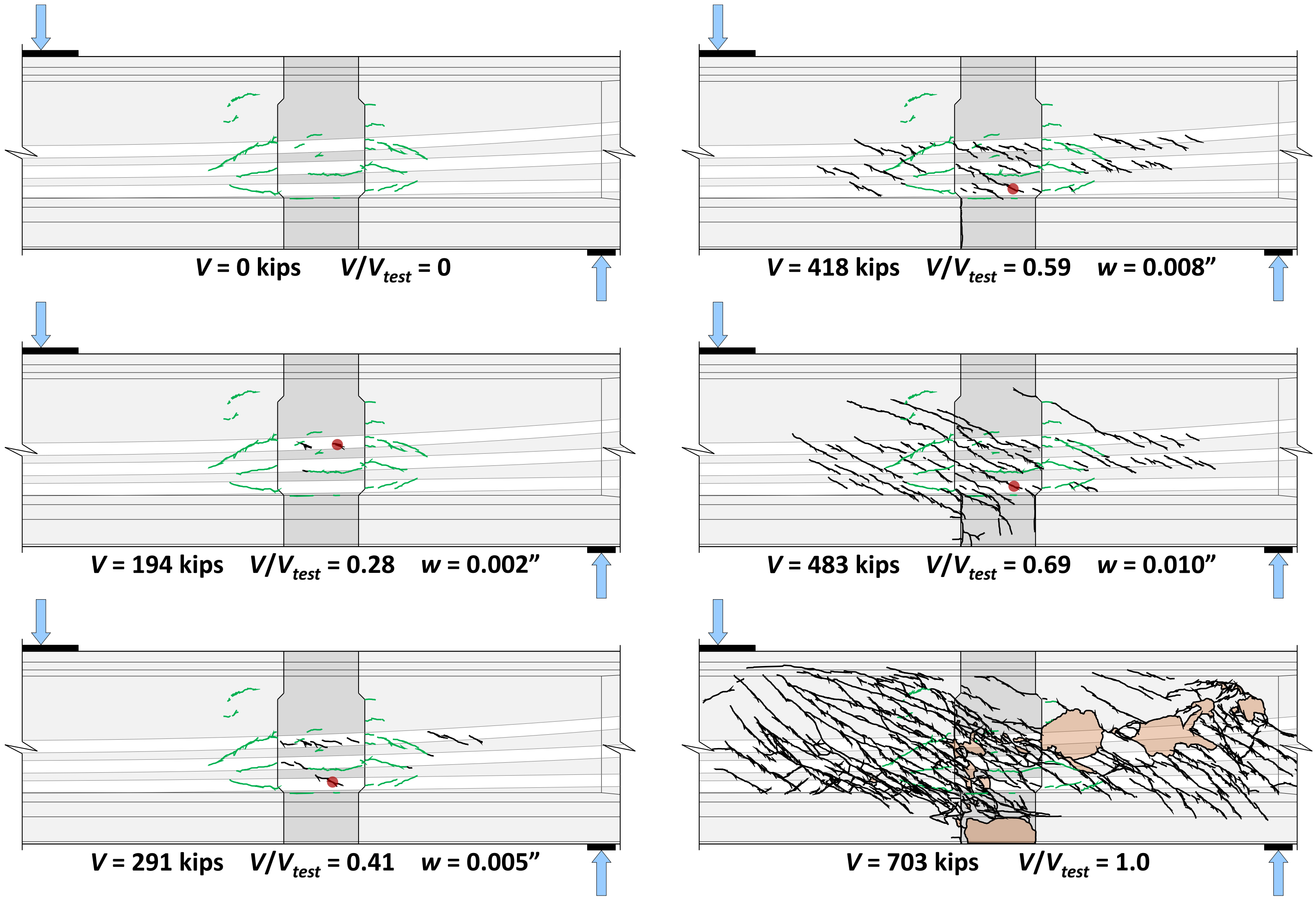


Figure 5.13: Summary of cracking behavior - Test Girder 2

5.4 EVALUATION OF STRENGTH BEHAVIOR

The following sections describe the behavior of the test girders up to the occurrence of shear failure. As described in Chapter 4, various sensor types were monitored during the tests to capture the behavior of the specimens. Localized displacements and deformations at the splice regions were of particular interest during the experimental program. Selected data are therefore presented to provide an overview of the splice region behavior. The maximum shear forces experienced by the girders are then compared to the calculated strengths according to sectional shear provisions.

5.4.1 Vertical Displacements at Splice Region

Relative displacements between the precast concrete segments and the CIP splice region of each girder were monitored during the tests by several linear potentiometers installed within the test span. Three linear potentiometers mounted to floor stands were placed on each side of the girders at the splice region as shown in Figure 5.14. The total shear force acting at the critical section is plotted against the displacements captured by the sensors in Figures 5.15 and 5.16 for the two test specimens. The plots were created by averaging the displacements measured by each pair of potentiometers located on opposite sides of the girders. One of the linear potentiometers located at the center of the splice region, however, did not provide complete data during either of the load tests. The displacement at this location is therefore plotted for only one sensor. Furthermore, displacements after the maximum force was reached were not accurately captured by this remaining potentiometer. Please note the placement of the sensors (north versus south) in relation to the load and support locations illustrated in Figure 5.14. It should be mentioned that the measured values were the result of several actions such as beam bending, splitting of concrete cover, and any localized differential displacements at the splice region interfaces that occurred.

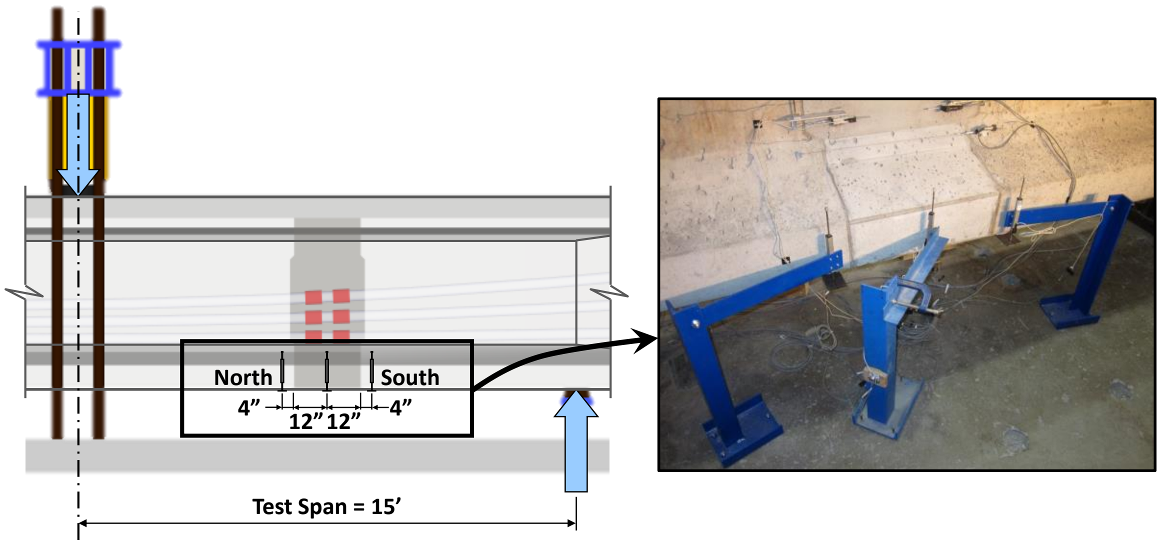


Figure 5.14: Linear potentiometers for measuring vertical displacements at the splice region

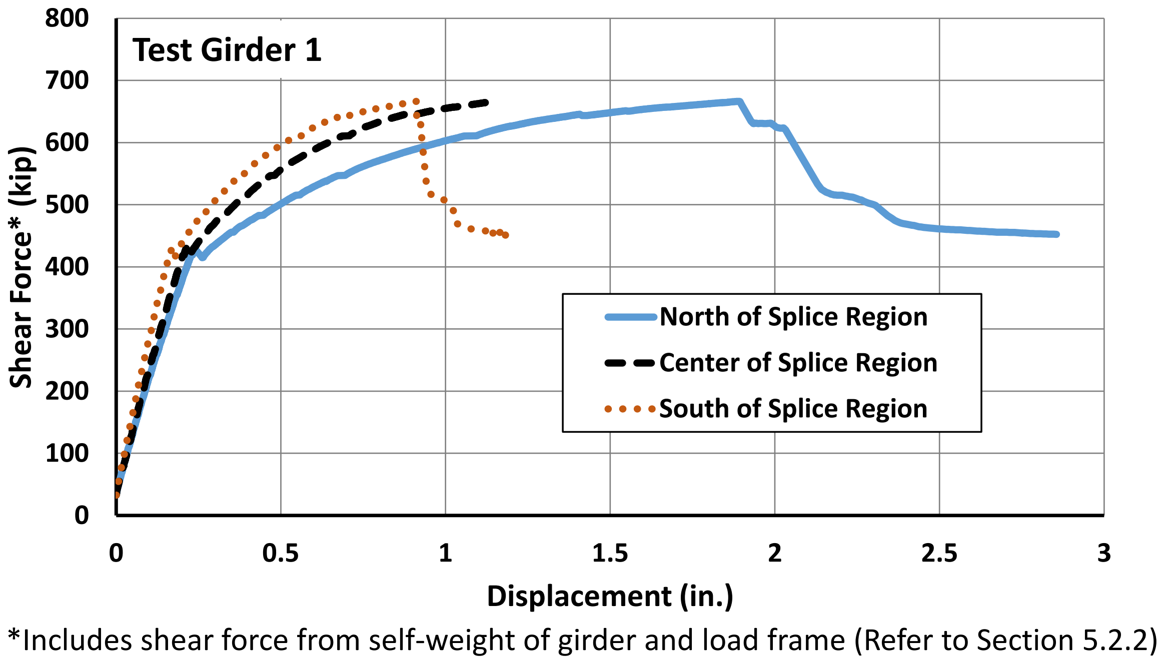


Figure 5.15: Measured vertical displacements at splice region of Test Girder 1

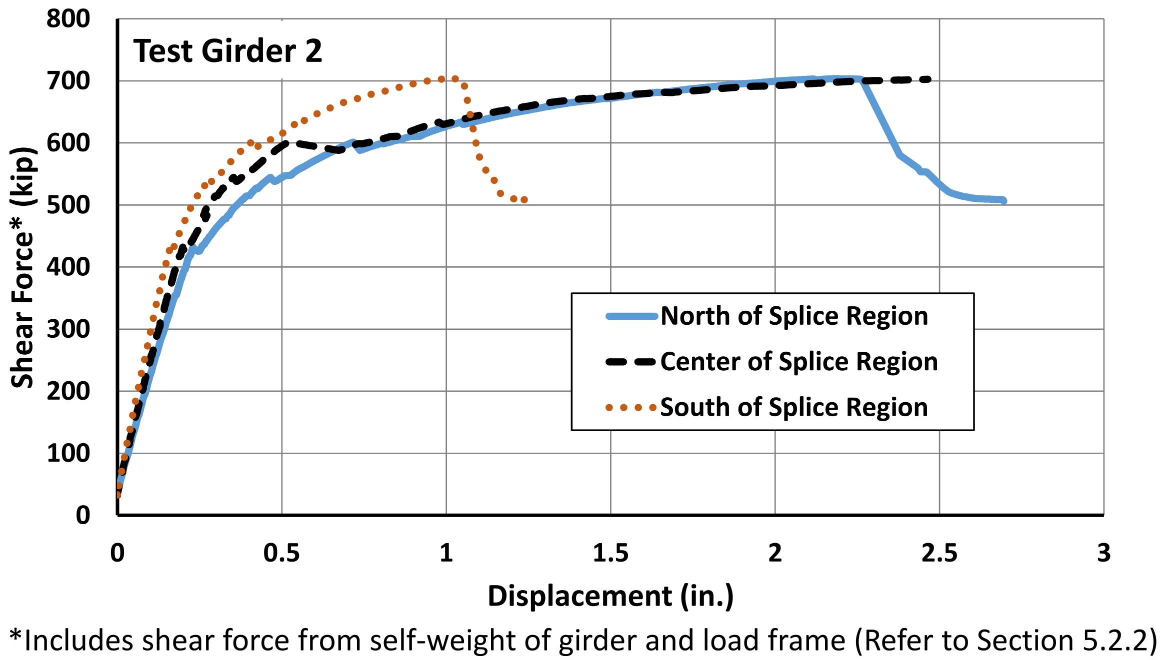
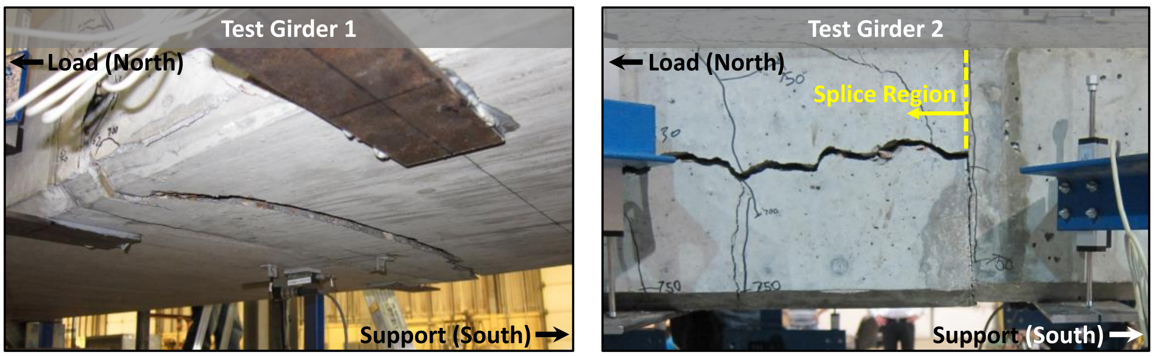


Figure 5.16: Measured vertical displacements at splice region of Test Girder 2

Considering the displacement data in Figures 5.15 and 5.16, the plots for both girders are fairly linear prior to the opening of flexural cracks at a shear force of approximately 430 kips (see Section 5.4.2). The deviation between the curves increases following this event. High levels of cracking and distress at the bottom flange of the test girders as the failure load was approached contributed to relative differences between the measured displacement values, as shown in Figure 5.17.



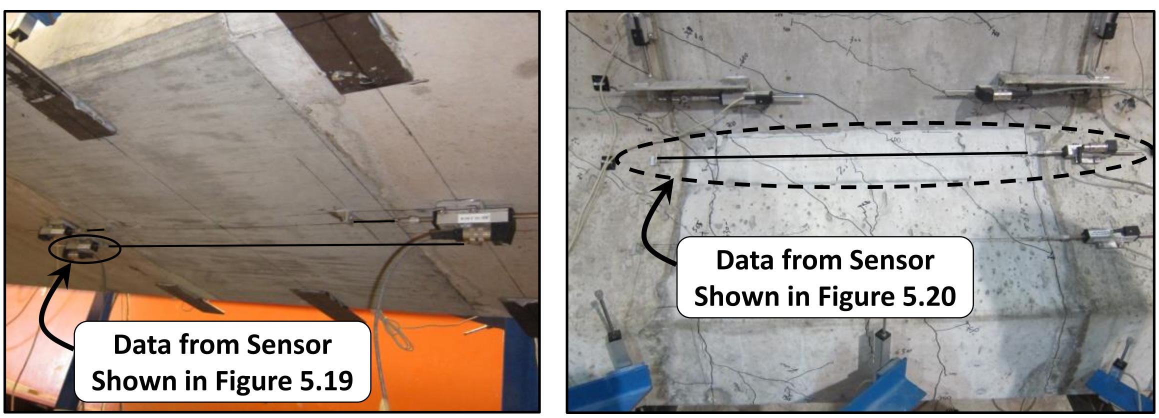
(a) *(b)*

Figure 5.17: Distress in the bottom flange at the splice region – (a) Test Girder 1 with V = 611 kips (0.92V_{test}); (b) Test Girder 2 with V = 610 kips (0.87V_{test})

5.4.2 Flexural Behavior of Splice Region

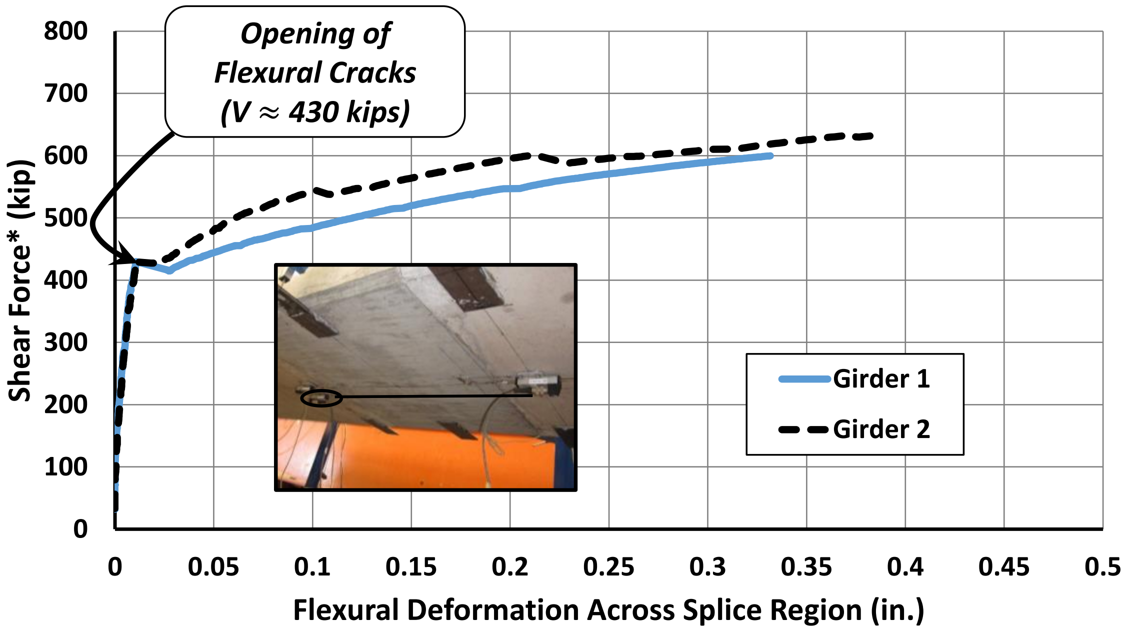
During the two load tests, high flexural demands were imposed on the girders, causing flexural cracks to form. All flexural cracking along the bottom flange was concentrated in the vicinity of the splice regions. For the first test girder, the opening of a flexural crack was accompanied by a slight drop in load and a loud popping sound at a shear force of 430 kips. The crack was located near the north interface between the long precast segment and the splice region (i.e., near the critical section). No flexural cracks were noted prior to this event. During the second load test, faint popping noises at a shear force of approximately 395 kips indicated the formation of a flexural crack near the critical section. Then, at a shear force of 432 kips, a slight drop in load accompanied additional flexural cracking at the splice region.

Linear potentiometers were installed to capture the opening of flexural cracks at the splice regions of the test girders, as shown in Figure 5.18. Strings mounted to each girder using small aluminum angle pieces were attached to the linear potentiometers shown in the figure to measure the flexural deformations of the member at the splice region. The displacements measured by the linear potentiometer installed on the bottom surface of the girders and indicated in Figure 5.18(a) are plotted in Figure 5.19 for both test specimens up to a shear force of 90 percent of V_{test}. Similarly, data from the potentiometer located near the top of the bottom flange of the girders, shown in Figure 5.18(b), are plotted in Figure 5.20 up to the maximum shear force. Please note that the string attached to both sensors extended across the entire splice region.



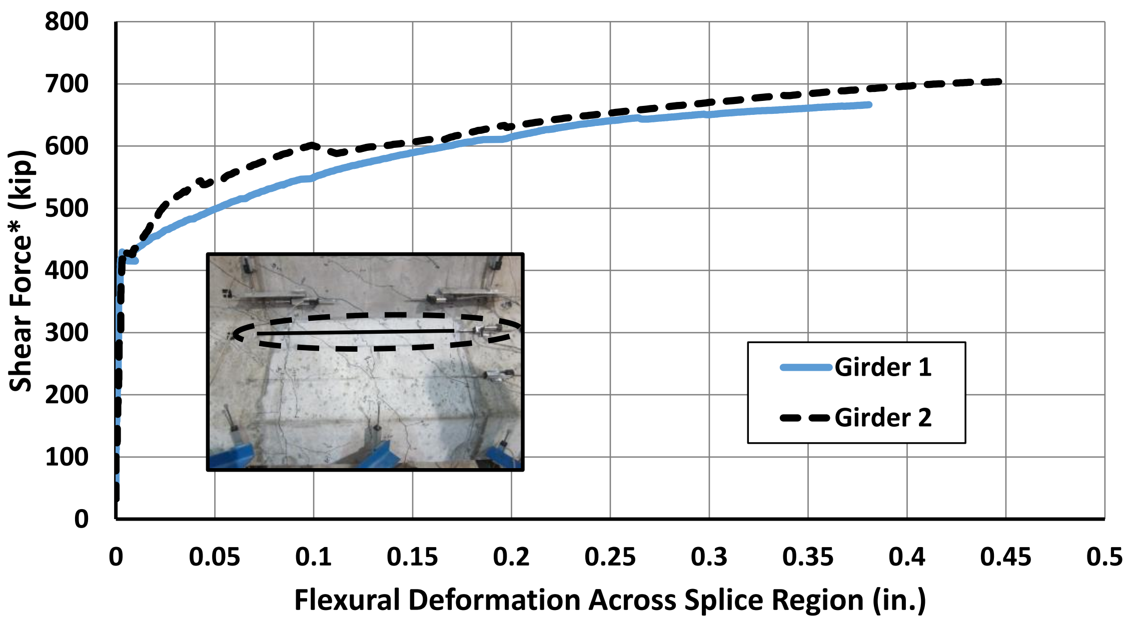
(a) *(b)*

Figure 5.18: Linear potentiometers measuring flexural deformations – (a) on the bottom surface; (b) at the bottom flange



*Includes shear force from self-weight of girder and load frame (Refer to Section 5.2.2)

Figure 5.19: Flexural deformations across bottom of splice region (plotted to $0.9V_{test}$)

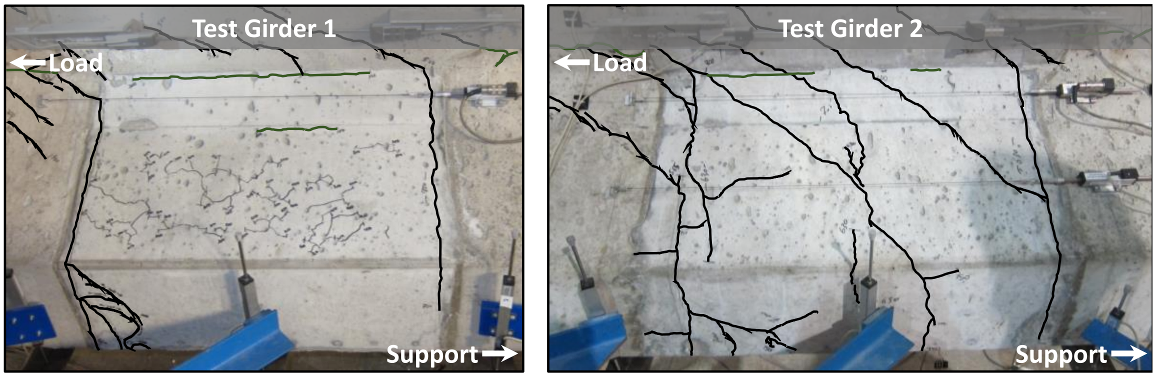


*Includes shear force from self-weight of girder and load frame (Refer to Section 5.2.2)

Figure 5.20: Flexural deformations across splice region at top of bottom flange (plotted to V_{test})

In Figure 5.19, the plots of the data from the linear potentiometers attached to the bottom surface of the test specimens indicate a similar behavior between the two girders. In fact, an abrupt opening of flexural cracks occurred at almost exactly the same load for both specimens (i.e., at a shear force of approximately 430 kips). Although relatively minor flexural cracking was noticed at a shear force of 395 kips for Test Girder 2, the primary flexural cracking began at nearly the same load for both specimens. The plots shown in Figure 5.20 confirm this behavior, revealing similarities between the behaviors of the test girders at the splice regions.

Although the overall flexural deformations across the splice regions indicate comparable behaviors between the test girders, the distribution of cracks within the bottom flanges of the two specimens were notably different. The longitudinal interface reinforcement extending from the precast segments into the splice regions (refer to Section 4.9.4) had a significant impact on the cracking behavior of the girders. To provide a comparison, the bottom flange at the splice region of each girder is shown in Figure 5.21 at a shear force of approximately 515 kips. (Please note that the small cracks marked on the top surface of the bottom flange of Test Girder 1 in Figure 5.21(a) are shrinkage cracks that existed before the start of the test.) The cracks in the bottom flange of Test Girder 1 are concentrated at the interfaces between the splice region and the precast segments. Test Girder 2, however, displays cracks that are distributed across the length of the splice region. The additional longitudinal reinforcement within the bottom flange of the second test girder prevented the cracking from concentrating at the splice region interfaces.



V = 515 kips (= 0.77V_{test})

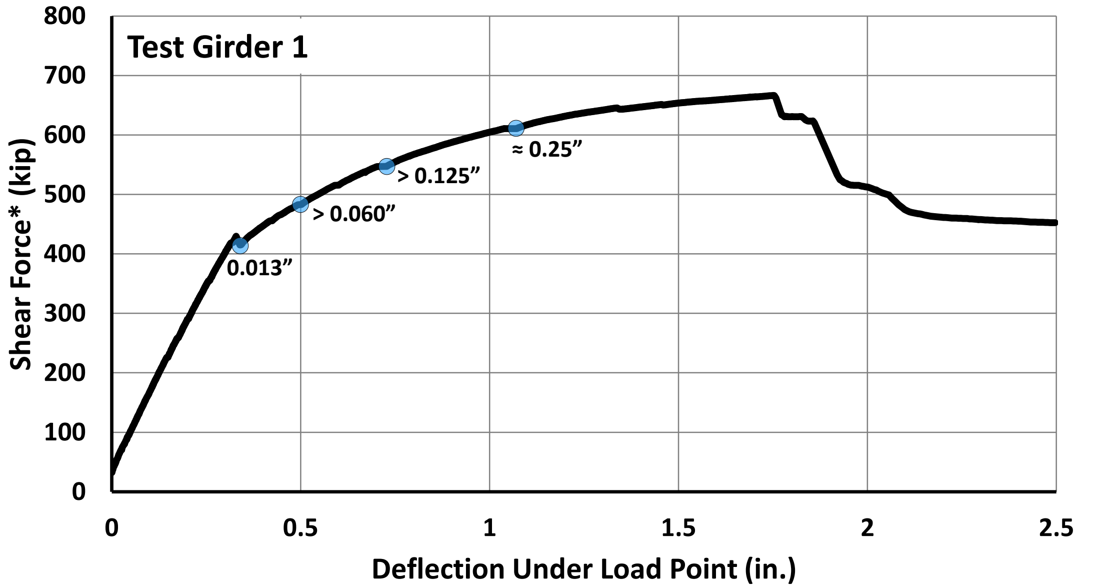
(a)

V = 514 kips (= 0.73V_{test})

(b)

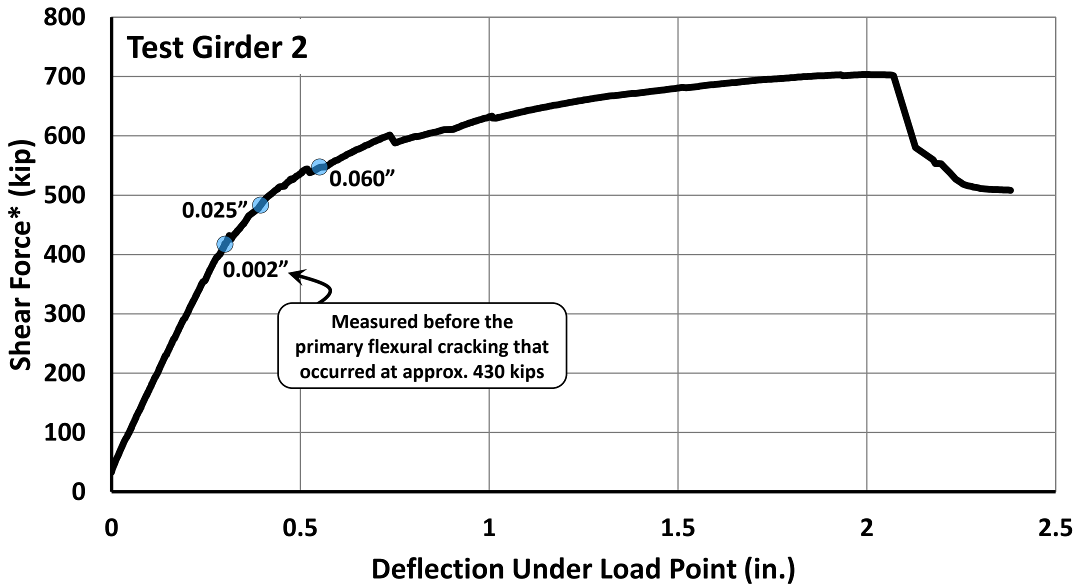
Figure 5.21: Cracking in the bottom flange at the splice region – (a) Test Girder 1; (b) Test Girder 2

The differences in the longitudinal interface reinforcement of the test girders also resulted in a notable distinction between measured flexural crack widths. More specifically, the width of the largest flexural crack, located near the critical section for both specimens, was significantly different for the two girders. To quantify the differences between the two specimens, the load-deflection plots of the girders are repeated in Figures 5.22 and 5.23 with the measured width of the largest flexural crack in the bottom flange labeled at various points during the tests. The plots indicate that the width of the flexural crack near the critical section of the first test girder was consistently larger than the corresponding crack of Test Girder 2, again demonstrating the improved flexural cracking behavior due to the additional interface reinforcement. It should be noted that the flexural behavior at the CIP splice regions of spliced girders in the field will depend on the specific details of the structure, such as the location of the post-tensioning tendons within the depth of the member. For example, if the splice region located in a positive moment region of a field structure is subjected to high flexural demands, the tendons are expected to be positioned within the bottom flange at this location. The crack widths indicated in Figures 5.22 and 5.23 are presented to reveal the effect of the interface reinforcement observed during the girder tests and may not represent the actual flexural crack widths experienced by field structures.



*Includes shear force from self-weight of girder and load frame (Refer to Section 5.2.2)

Figure 5.22: Load-deflection plot with flexural crack widths – Test Girder 1



*Includes shear force from self-weight of girder and load frame (Refer to Section 5.2.2)

Figure 5.23: Load-deflection plot with flexural crack widths – Test Girder 2

Measured strains in the longitudinal interface reinforcement of both girders are consistent with the visual observations noted during the tests. Strains in the reinforcement indicated in Figure 5.24(a) are plotted in Figure 5.24(c) up to 70 percent of the maximum shear force for both girders. (The strain gauges did not produce accurate measurements as loads higher than this value were approached.) The data were gathered from strain gauges installed on the rebar near the splice region interface at the critical section (see Figure 5.24(b)). The bottom flange illustrated in Figure 5.24(a) includes the interface reinforcement of both girder specimens (i.e., Bars K of Test Girder 1 and Bars J of Test Girder 2). For each girder, data from a strain gauge installed on one of the bars within each layer labeled in the figure (i.e., “upper” or “lower” layer) are presented in Figure 5.24(c). Please note that an initial compressive strain in the reinforcement that was introduced and monitored during the post-tensioning operations is included in the plots.

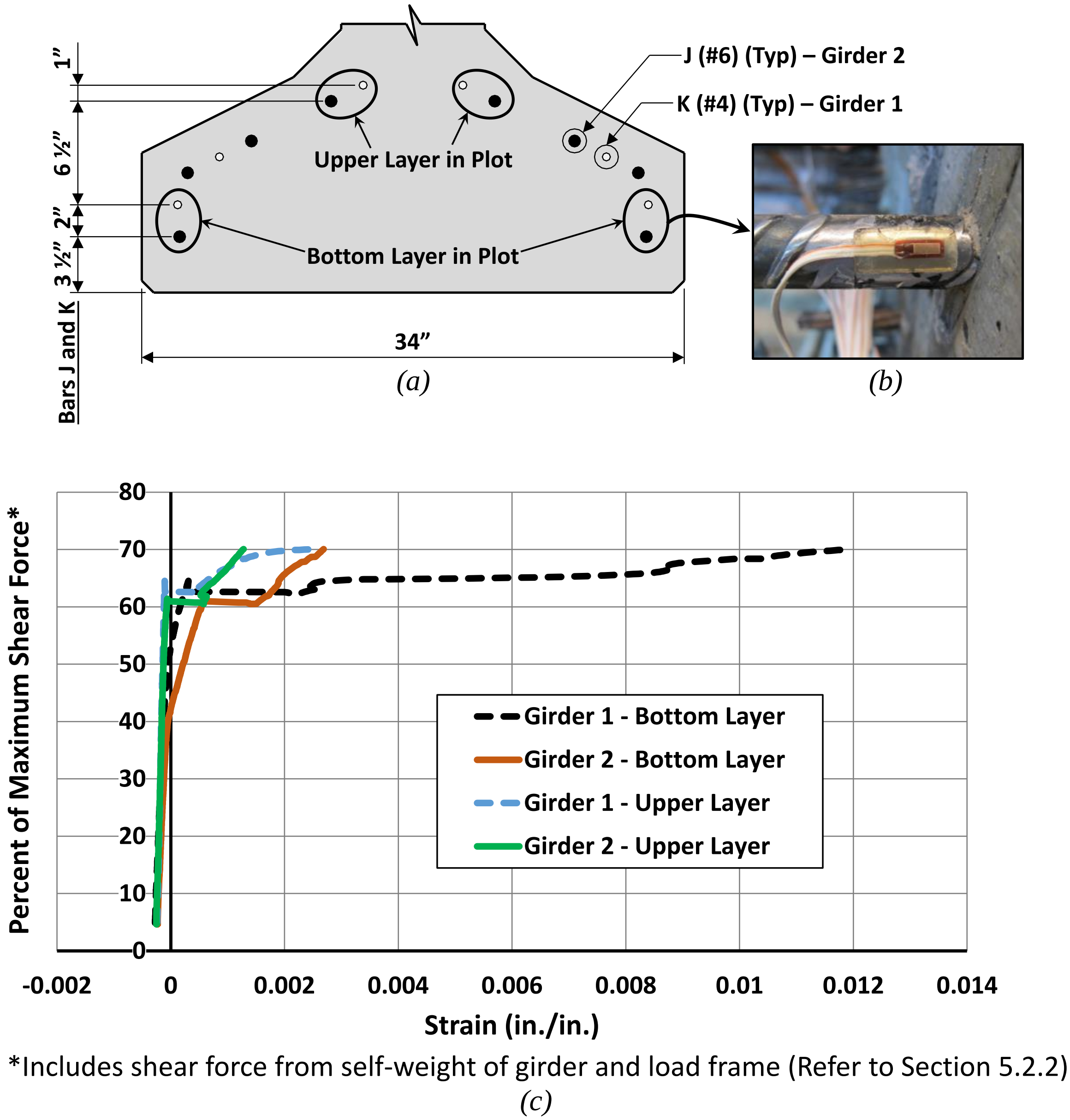


Figure 5.24: Strain in longitudinal interface reinforcement – (a) bars monitored; (b) location of gauges; (c) strains measured near the splice region interface

The strain data agree with the observed crack distribution within the bottom flanges of the two test girders. As the applied load increased, the reinforcement within the first test girder tended to experience higher strains near the splice region interface compared to the bars within Test Girder 2. This corresponds with a wider crack near the interface of Test Girder 1 and therefore less distribution of cracks through the length of the splice region.

Although the interface reinforcement did affect the splice region behavior after the initiation of flexural cracking within the bottom flange, the formation of flexural cracks at the splice region interface can be prevented by ensuring that the tensile stress in the extreme fiber of the girder due to applied loads does not overcome the compressive stress imposed by the post-tensioning force. As an example, using gross cross-sectional properties, reduced as necessary to account for empty ducts, along with the stress in the post-tensioned strands at the completion of the post-tensioning operations (refer to Table 4.7), the compressive stress at the bottom fiber of Test Girder 1 at the critical section is calculated to be 1.60 ksi. Considering the gross cross-sectional properties of the composite girder (i.e., the girder plus the deck), the moment required to overcome this compressive stress, including the moment from the self-weight of the specimen, is 36,513 kip-in. This moment corresponds with a shear force of 359 kips, 83 percent of the shear force when flexural cracks were noted (430 kips). This provides evidence that flexural cracking can be prevented under service-level design loads. When designing a spliced girder bridge, however, long-term effects on the cracking moment should also be considered.

According to Article 5.14.1.3.2d of AASHTO LRFD (2014), concrete stress limits for segmentally constructed bridges should also be applied at the splice regions (i.e., "joints") of spliced girders. Therefore, no tension is allowed in the precompressed tensile zone at the splice region, as considered in the preceding discussion, unless auxiliary reinforcement capable of carrying the longitudinal tensile force at a stress of $0.5f_y$ is provided. In this case, a maximum tensile stress of $0.0948\sqrt{f'_c}$ is permitted. However, the discussion of flexural cracking at post-tensioned in-span CIP splice regions is limited to the results of the two spliced girder tests. Moreover, the possibility of significant flexural cracking in spliced girder bridges may exist in some cases, especially when subjected to overloads. Designing spliced girders to experience no tension under service-level loads at the splice regions is therefore recommended. Additional testing may provide the information needed to refine the stress limits applied at CIP splice regions.

5.4.3 Strain in Stirrups

Strains in the stirrups (i.e., shear reinforcement) located within the splice regions were monitored during testing to identify any notable trends and to relate the strains to the cracking behavior of the girder webs. The placement of strain gauges on the shear reinforcement is illustrated in Figure 5.25. The two stirrups located at the center of the splice regions were instrumented. Three strain gauges were placed on each stirrup to correspond with the locations of the post-tensioning ducts. A fourth strain gauge was also installed within Test Girder 2 to monitor rebar strains above the ducts. The strains measured up to the maximum shear force by the four gauges installed on a stirrup within the splice region of Test Girder 2 are presented in Figure 5.26. Although some variations existed between the strains measured for each stirrup, the trends indicated in the figure were fairly typical.

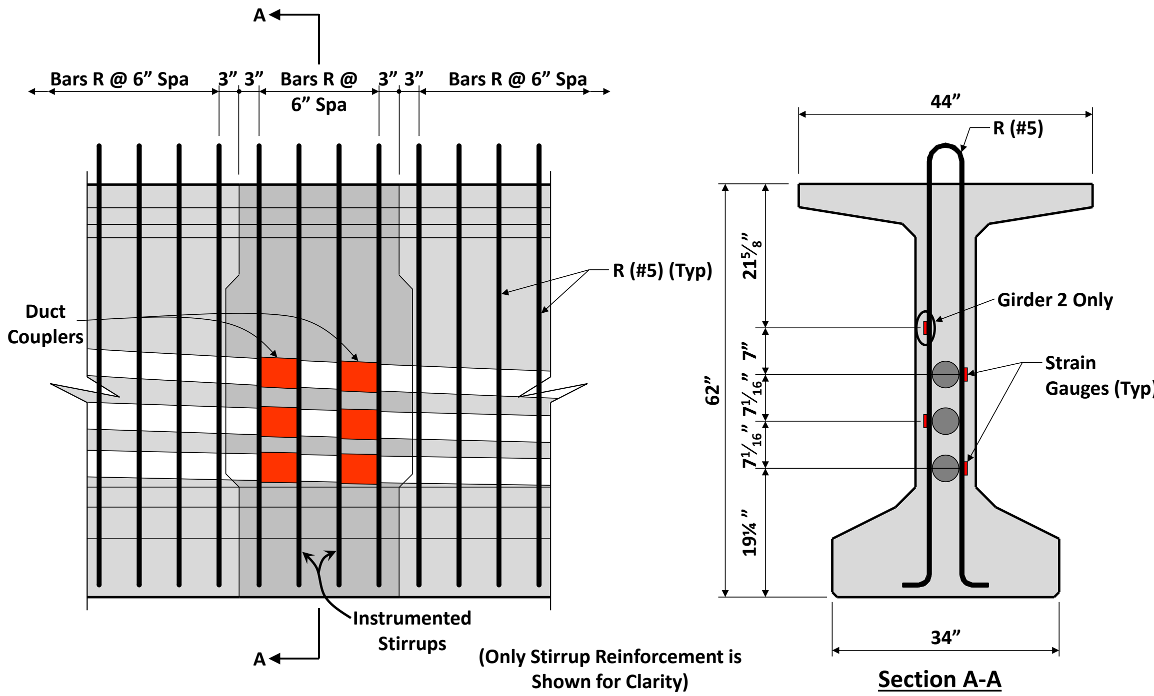


Figure 5.25: Strain gauges installed on stirrup reinforcement within the splice region

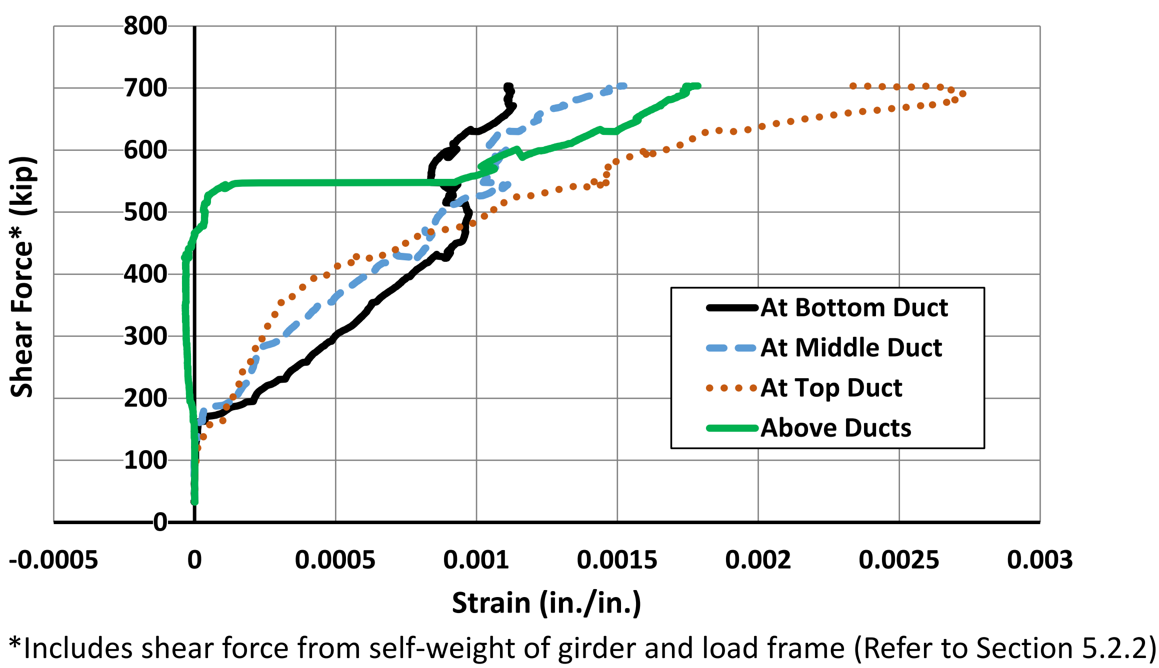


Figure 5.26: Strains in stirrup reinforcement within the splice region (plotted to V_{test}) – Test Girder 2

Examining the data plotted in Figure 5.26 results in a few interesting observations. First, the stirrup reinforcement experienced notable tensile strains near the locations of the ducts at a significantly lower shear force than at the strain gauge placed above the ducts. The development of tensile strains near the ducts corresponds with the observance of the first web-shear cracks at a shear force of 194 kips (refer to Section 5.3). Furthermore, as the girder approached failure, the reinforcement was strained highest at the location of the top post-tensioning duct. This is expected based on the shear-compression failure mechanism characterized by concrete crushing in the vicinity of the top duct.

5.4.4 Comparison of Tested Capacities to Calculated Strengths

The experimental shear capacities of the two spliced girder test specimens were compared to the calculated strengths based on the general shear procedure of Article 5.8.3.4.2 of AASHTO LRFD (2014) and the proposed shear design procedure introduced in Moore (2014). The location of the critical section for the evaluation of the shear strength calculations was provided in Section 5.2.2. The calculated shear strengths, V_n, and the experimental shear capacities, V_{test}, are summarized in Table 5.1.

Table 5.1: Summary of Experimental Capacities and Calculated Strengths (Using Duct Diameter of 4 in.)

Test Specimen	V_{test} (kips)	AASHTO LRFD (2014) General *Article 5.8.3.4.2*		Proposed Procedure *Moore (2014)*	
		V_n (kips)	V_{test}/V_n	V_n (kips)	V_{test}/V_n
Girder 1	666	638	1.04	563	1.18
Girder 2	703	656	1.07	573	1.23

To account for the discontinuity in girder webs due to the presence of post-tensioning ducts, the calculated strength based on the AASHTO LRFD (2014) general shear procedure considers a web width reduction of one-quarter the diameter of grouted ducts, as stated in Article 5.8.2.9 of the specifications. The shear design procedure introduced in Moore (2014) and summarized in Appendix E proposes that the gross web width, b_w, be used within the AASHTO LRFD general shear procedure and a quadratically decreasing strength reduction factor, λ_{duct}, be applied to the shear resistance provided by the transverse reinforcement, V_s.

To calculate the shear strength based on both the AASHTO LRFD (2014) general shear provisions and the proposed procedure, it was first necessary to define the values used within the design equations. Considering that the critical section is located at the interface between the precast girder segment and the CIP splice region, the concrete compressive strength, f'_c, used in the calculations was governed by the lower-strength splice region concrete. Furthermore, the value of f_{po} within Equation 5.8.3.4.2-4 of AASHTO LRFD (2014), presented as Equation 2.8 in Chapter 2, was taken as the average tensile stress in the post-tensioning tendons after the anchorages were set (refer to Section 4.14.1). Alternatively, the value of f_{po} could have been taken as the stress in the tendons at the time of testing, as some past researchers have assumed. The additional prestressing losses between the completion of each post-tensioning operation and the load test were small and would result in only minor changes to the calculated shear strengths. The post-tensioning stress in each tendon and the concrete compressive strengths used for the shear calculations were presented in Tables 4.6 and 4.7.

The mild longitudinal interface reinforcement was not considered to contribute to the strength of the girders. As explained in the following section (Section 5.5), the girder behavior provided evidence that the interface reinforcement was not fully effective at the ultimate state of the girder specimens. Therefore, it is recommended that the contribution of the mild interface reinforcement not be considered in flexural or sectional shear strength calculations, as was assumed for the calculated strengths presented in Table 5.1.

The only steel considered effective within the splice regions at the ultimate state was the post-tensioning tendons and the transverse shear reinforcement (i.e., stirrups). As previously stated, any effect of the vertical reinforcement added within the end region of the precast segments to provide splitting resistance was not included in the shear strength calculations at the critical section (refer to Section 4.6).

The shear strength ratios, V_{test}/V_n, presented in Table 5.1 indicate that both sectional shear procedures can be applied at the splice region interface of the test specimens; the shear strength ratio is greater than unity in all cases. The proposed shear design procedure, however, was slightly more conservative, with the lowest V_{test}/V_n ratio having a value of 1.18.

As noted in Section 4.8.1, any effects due to the relatively large size of the plastic duct couplers within the splice region were of particular interest. It should be noted that a duct diameter of 4 in. was assumed for all shear strength calculations presented in Table 5.1. (For the particular ducts used in the specimens, the specified duct diameter of 4 in. actually corresponds with the inner diameter of the ducts.) Using the outer diameter of the coupler as opposed to the 4-in. duct diameter does, of course, result in more conservative design calculations. The calculated shear strengths at the critical section using a coupler diameter of 4 $^{13}/_{16}$ in. (refer to Figure 4.13 for coupler dimensions) are provided in Table 5.2. The strengths calculated using the current AASHTO LRFD (2014) general shear procedure change only slightly compared to the values presented in Table 5.1. The increase in the diameter has a more significant impact on the strengths calculated using the proposed procedure. Although additional conservatism may be desirable as discussed in Section 7.2, the exact diameter of the coupler that will be used in the field may not be known by the engineer at the design stage. Requiring the use of the coupler diameter in design calculations, therefore, may not be practical.

Table 5.2: Summary of Experimental Capacities and Calculated Strengths (Using Outer Coupler Diameter of 4 13⁄16 in.)

Test Specimen	V_{test} (kips)	AASHTO LRFD (2014) General *Article 5.8.3.4.2*		Proposed Procedure *Moore (2014)*	
		V_n (kips)	V_{test}/V_n	V_n (kips)	V_{test}/V_n
Girder 1	666	637	1.05	534	1.25
Girder 2	703	655	1.07	539	1.30

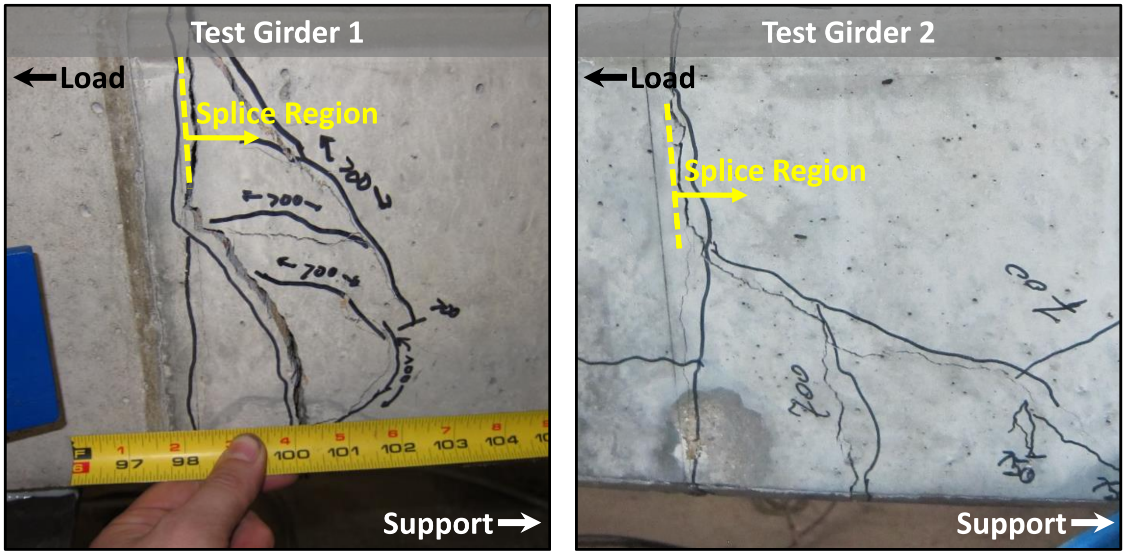
To further investigate any possible effect of the duct couplers on the shear behavior of the test girders, the video footage recorded during the tests was examined at the moment of failure to determine the location of the initial concrete crushing that led to the reduction in the load-carrying capacity of the specimens. The failure of the girders was captured at 30 images per second. For the first test girder, only one side of the specimen was successfully recorded at the moment of failure. The concrete crushing leading to the drop in the load-carrying capacity of the girder is believed to have initiated on the side of the girder that was not recorded. The failure of the second girder was successfully captured from both sides of the specimen. Examining each captured image does not, however, provide a clear indication of whether the concrete crushing at the moment of failure initiated within the splice region or in the precast segment. Therefore, no conclusive evidence suggests that the duct couplers had a significant effect on the girders at failure.

Finally, it is important to note that the maximum shear stress limit of $0.25f'_c$ in Article 5.8.3.3 of AASHTO LRFD (2014) did not govern the calculated shear strength for either test girder. More detailed shear strength calculations for the specimens are provided in Appendix C.

The maximum moment acting at the critical section of each girder during the load tests was equal to 90 percent and 91 percent of the nominal flexural resistance, M_n, for the first and second specimens, respectively. The flexural resistance was calculated using a non-linear sectional analysis program. A 1-in. offset of the post-tensioning tendons in the ducts was assumed, and the strength of the splice region concrete was assigned to the girder section below the deck. Based on the behavior generally observed during flexural tests on reinforced concrete members, experimental flexural capacities are typically greater than calculated strengths.

5.5 Influence of Longitudinal Interface Reinforcement

The primary variable between the two spliced girder test specimens was the mild longitudinal interface reinforcement extending from the precast segments into the splice region. As previously discussed, the additional interface reinforcement provided in Test Girder 2 resulted in a more distributed crack pattern in the bottom flange when compared to the behavior of the first test girder. The flexural cracks near the splice region interface that is located at the critical section are shown in Figure 5.27 for both test girders after reaching a shear force of 547 kips. The photographs further illustrate the concentration of cracking near the splice region interface of Test Girder 1, resulting in a wider crack opening at this location compared to Test Girder 2. In the figure, the flexural crack in Test Girder 1 has a width of at least 0.125 in., while the crack in the second test girder was measured to be 0.060-in. wide. Therefore, additional interface reinforcement resulted in a notable difference in the flexural cracking behavior at the splice region.

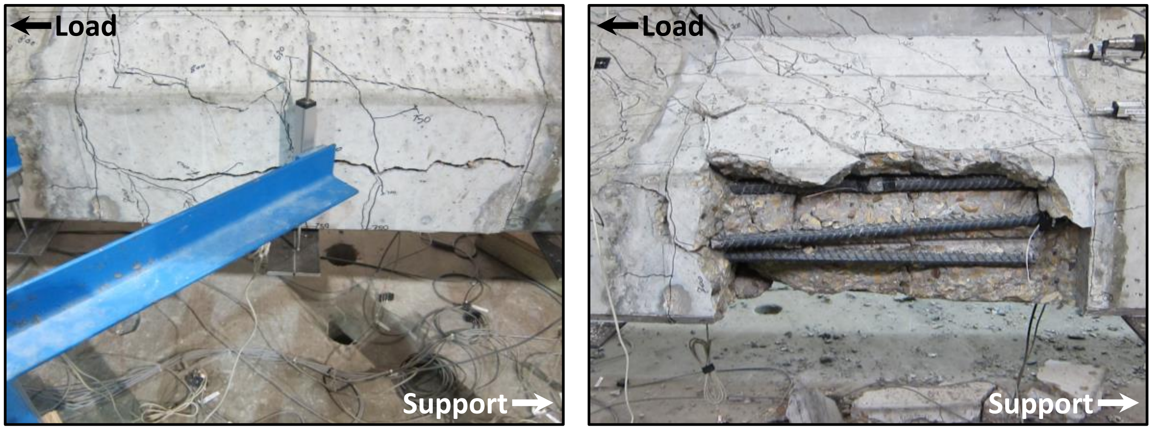


***V* = 547 kips (= 0.82V_{test})**
(a)

***V* = 547 kips (= 0.78V_{test})**
(b)

Figure 5.27: Flexural cracking near the splice region interface – (a) Test Girder 1; (b) Test Girder 2

The appearance of the bottom flange of the second test girder at both a shear force of 0.87V_{test} and after the occurrence of shear failure is presented in Figure 5.28. As the ultimate shear force was approached, horizontal cracks corresponding with the locations of longitudinal interface bars opened within the bottom flange (Figure 5.28(a)). Upon further loading, concrete cover began to spall, as indicated in Figure 5.28(b).



V = 610 kips (= $0.87V_{test}$)
(a)

Post-Failure
(b)

Figure 5.28: Bottom flange at splice region of Test Girder 2 – (a) at a shear force of $0.87V_{test}$; (b) post-failure

The behavior shown in Figure 5.28 provides evidence that longitudinal interface bars extending into the splice region should not be considered effective at the ultimate state. It is therefore recommended that the contribution of interface reinforcement be conservatively neglected in flexural and sectional shear strength calculations, as assumed for the values presented in Section 5.4.4.

5.6 SUMMARY

In this chapter, the results and observations from the load tests conducted on the two spliced girder test specimens were presented with special focus placed on the behavior of the cast-in-place splice regions. Both test girders exhibited a shear-compression failure mechanism characterized by crushing of the web concrete in the vicinity of the top post-tensioning duct. The web crushing observed during failure of each specimen was not localized at the splice region but extended across much of the test span.

Both the strength and service-level shear behaviors were also detailed, including the cracking behavior and deformations within the splice regions. The effect of the longitudinal interface reinforcement on the girder behavior after the formation of flexural cracks was evaluated. Comparisons of the experimental shear capacities of the girders with calculated strengths revealed that both the AASHTO LRFD (2014) general shear provisions and the proposed shear design procedure introduced in Moore (2014) provide conservative strength estimates for the test girders at the splice regions.

An in-depth study of interface shear transfer relating to CIP splice regions is presented in Chapter 6. Design recommendations based on the results of the splice region research program are then provided in Chapter 7.

Chapter 6. Interface Shear Transfer at Splice Regions

6.1 Introduction

The design of the splice regions of spliced girder bridges requires the ability to estimate the shear stresses that can be transferred across the splice region interfaces between precast concrete and cast-in-place (CIP) splice region concrete. The results of the industry survey described in Chapter 3 indicated that various shear interface details have been specified at the splice regions of existing bridges. At the same time, no studies focusing on interface shear transfer between precast girders and CIP splice regions have been identified. The shear-friction performance of the various interface details commonly specified at splice regions cannot therefore be directly compared.

To gain a better understanding of interface shear transfer at CIP splice regions, an experimental study was conducted to supplement the large-scale spliced girder tests. The results of eleven push-through tests conducted on reinforced concrete specimens are described in this chapter. The test results are analyzed to assess the performance of specimens with various interface details. The effects of different combinations of mild interface reinforcement and post-tensioning force on interface shear strength are also studied. Furthermore, the test results are used to evaluate current shear-friction code expressions. The research findings are then discussed in relation to spliced girder design.

6.2 Shear-Friction Experimental Program

6.2.1 Overview and Objectives

The two large-scale spliced girder tests described in the preceding chapters provided a means to study the sectional shear and flexural behavior of CIP splice regions. The girder tests, however, were not designed to evaluate shear-friction provisions, and interface shear failures did not occur. To gain a deeper understanding of interface shear transfer at CIP splice regions, an experimental program was developed to focus exclusively on the shear-friction behavior of specimens representing the webs of spliced I-girders.

Past shear-friction studies have typically been based on the performance of push-off tests such as those shown in Figures 6.1(a) and (b). Push-through tests illustrated in Figure 6.1(c), however, were identified as a more suitable approach to study the interface shear performance of the CIP splice regions of large spliced I-girders.

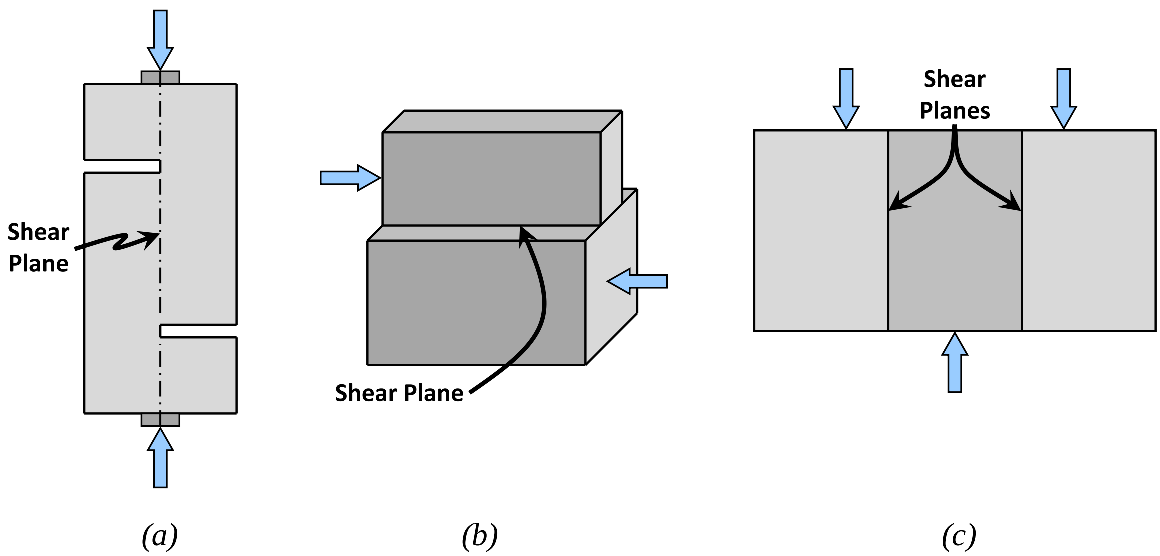


Figure 6.1: Shear-friction tests – (a) push-off type 1 (adapted from Mattock and Hawkins (1972)); (b) push-off type 2 (adapted from Bass, Carrasquillo, and Jirsa (1989)); (c) push-through

The primary objectives of the shear-friction experimental program can be divided into three components as follows:

(i) Evaluate the effect of various shear interface details on interface shear strength and behavior.

(ii) Determine the influence of mild interface reinforcement and the level of post-tensioning force on interface shear strength and behavior.

(iii) Evaluate current shear-friction code expressions using the results of the push-through tests.

The findings from the testing program were also extended to the CIP splice regions of spliced I-girders to supplement the large-scale experimental program described in the preceding chapters.

The shear-friction push-through tests were conducted and first presented by Massey (2014). Details from eleven tests of the original experimental program are presented in this chapter. Data from these tests have been reinterpreted and analyzed as described in Section 6.3.

6.2.2 Specimen Details

The push-through test specimens were designed to represent the webs of the two spliced I-girders tested during the splice region experimental program. The nominal thickness of the specimens therefore matched the girder web width, b_w, of 9 in. The specimens had a length of 72 in. and a depth of 36 in., as illustrated in Figure 6.2. Each specimen consisted of three segments, each with a length of 24 in. The two outer segments were cast at the same time (labeled as Cast #1 in Figure 6.2). At a later date, the inner segment was cast between the two outer segments (labeled as Cast #2 in Figure 6.2). The new concrete was cast directly against the surfaces of the outer segments. The reinforcement crossing the interfaces between the concrete segments was left extended from the outer segments prior to Cast #2 through the use of custom wooden formwork, as shown in Figure 6.3. All horizontal bars were embedded for at least the required development length, l_d, on each side of the interface in accordance with Equation 25.4.2.3a of ACI 318-14. The average time that elapsed between casting the inner and outer segments was 10 days.

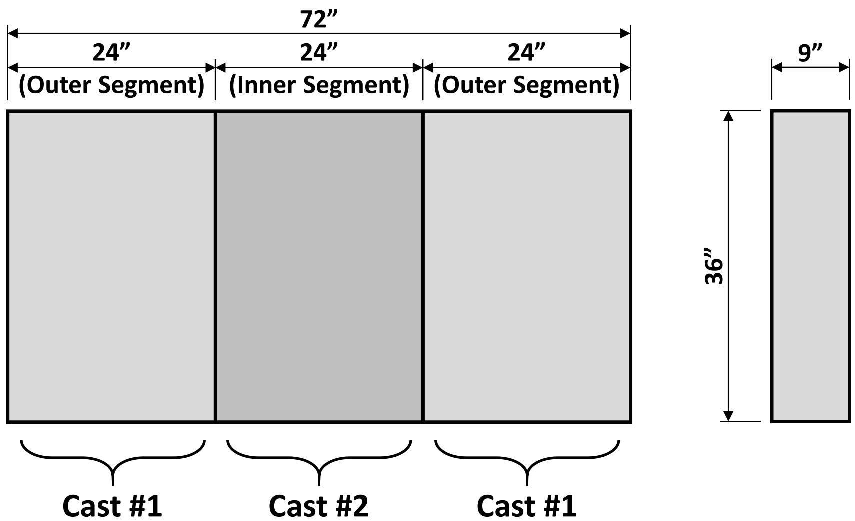


Figure 6.2: Push-through specimen geometry and casting scheme

Figure 6.3: Custom formwork for outer segments

The eleven specimens can be divided into two sets based on their experimental variables. Set 1 consists of five push-through specimens and was developed to study the effect of various shear interface details. Set 2 includes six test specimens that were designed to evaluate the influence of mild interface reinforcement and post-tensioning force on interface shear behavior.

The five specimens in Set 1 are shown in Figure 6.4. To focus on the performance of various shear interfaces, each specimen was fabricated with a unique surface detail at the interface between the inner and outer segments. The five interface types are identified as follows: smooth surface (Specimen 1-1), single 12-in. shear key (Specimen 1-2), two 6-in. shear keys (Specimen 1-3), 1-in. saw teeth (Specimen 1-4), and 2-in. saw teeth (Specimen 1-5). The reinforcement and external post-tensioning force applied to the specimens were control variables. The vertical reinforcement (see Figure 6.5) consisted of No. 3 closed stirrups spaced at 6 in. along the length of the specimens. Straight No. 4 bars were provided as longitudinal reinforcement. The reinforcement crossing the interfaces between the inner and outer segments alternated between the two sides of the cross-section, as shown at Section A-A in Figure 6.5. Thus, the interface reinforcement consisted of four No. 4 bars. The longitudinal reinforcement that was discontinued near the interfaces had a clear cover of 1 ½ in. at the ends of the bars. The external post-tensioning force applied to the Set 1 specimens was maintained at approximately 163 kips throughout each test, resulting in an average stress of 0.5 ksi over the cross-section of the specimens.

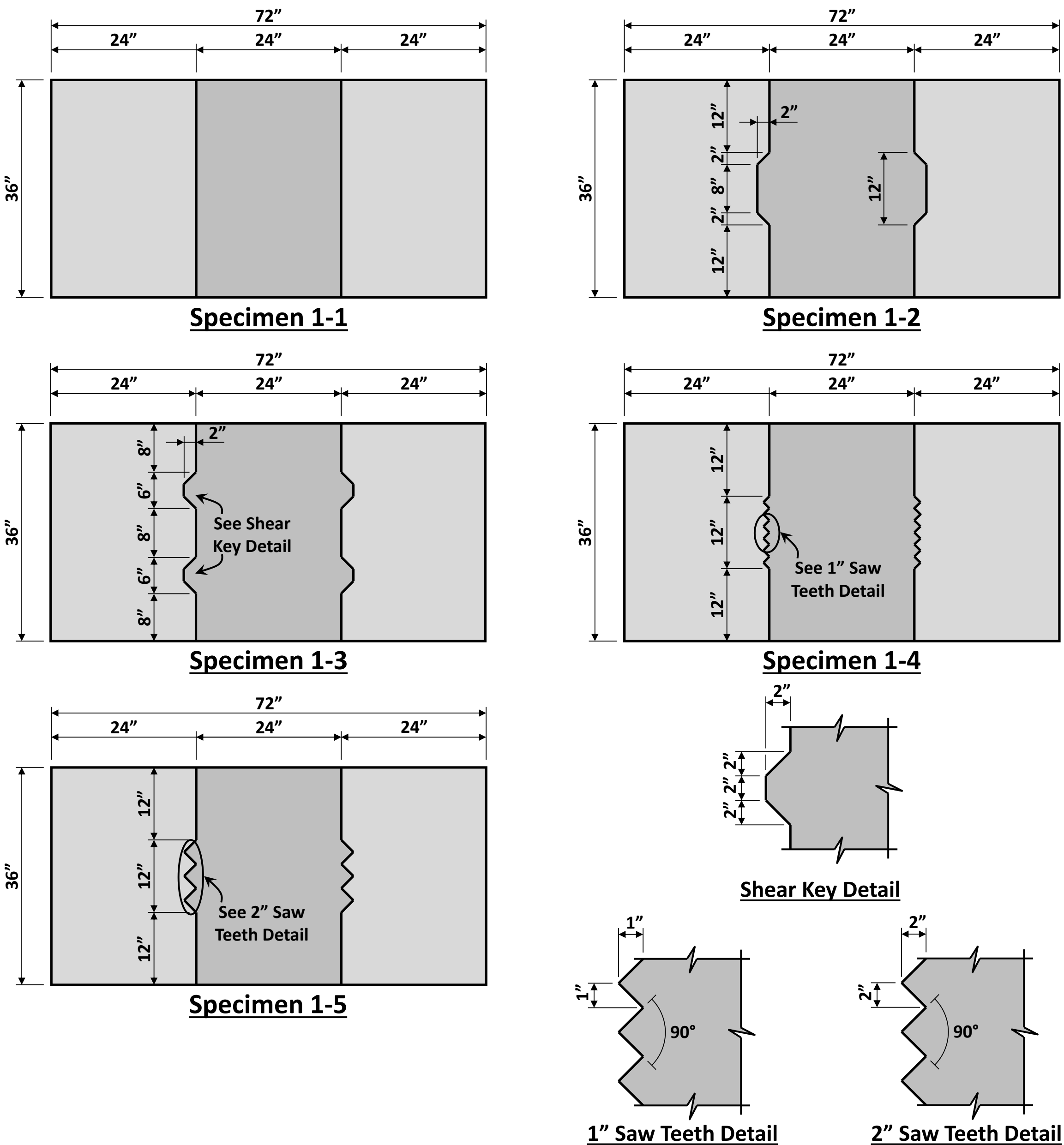


Figure 6.4: Set 1 shear interface details

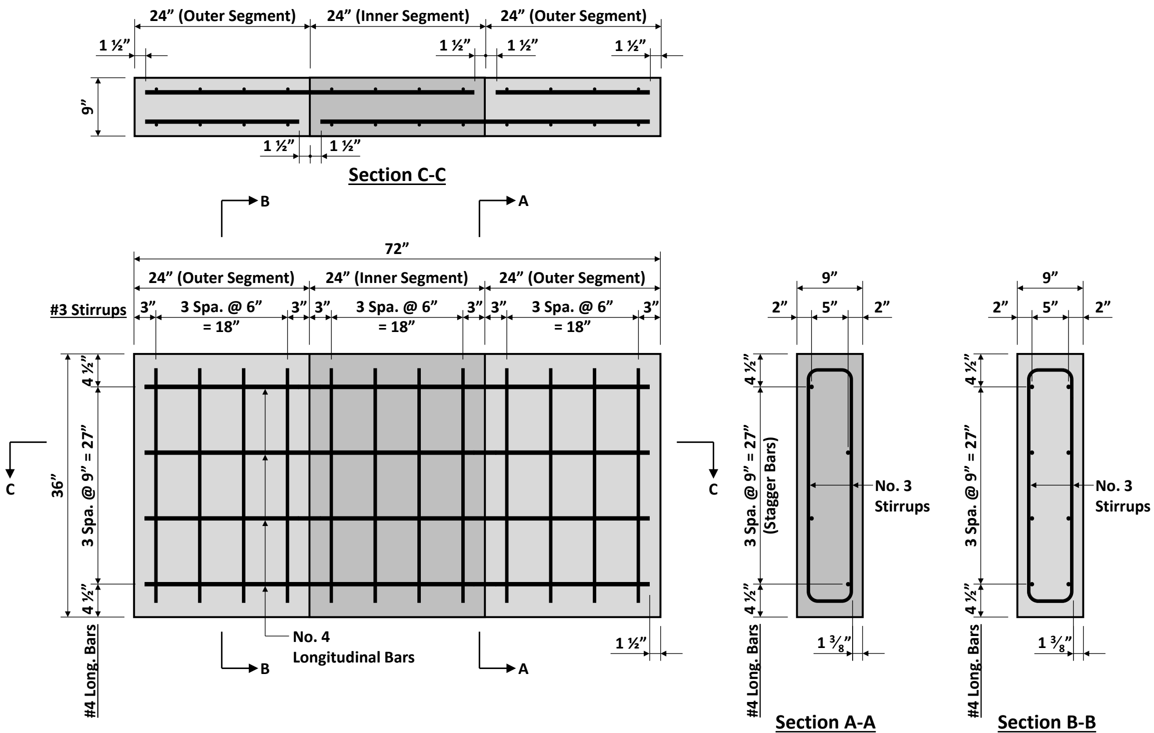


Figure 6.5: Reinforcement details of all Set 1 specimens and Specimens 2-4, 2-5, and 2-6 of Set 2

The primary experimental variables of the six specimens in Set 2 were the level of the external post-tensioning force applied to the specimens and the area of interface reinforcement crossing between the inner and outer segments. The shear interface of each Set 2 specimen was detailed with a single 10-in. long shear key as shown in Figure 6.6. The reinforcement crossing the interface between the inner and outer segments of Specimens 2-1 and 2-2 consisted of eight bars and is illustrated in Figure 6.7. No. 5 longitudinal bars were provided in Specimen 2-1, while No. 4 bars were provided in Specimen 2-2. No bars crossed the interfaces of Specimen 2-3, as shown in Figure 6.8; all longitudinal reinforcement was discontinued within each segment as illustrated. The interface reinforcement of the remaining three specimens (2-4, 2-5, and 2-6) consisted of four No. 4 bars, matching the rebar details of the Set 1 specimens presented in Figure 6.5.

Also consistent with Set 1, No. 3 stirrups spaced at 6 in. were provided in all six Set 2 specimens. The external post-tensioning force applied to each specimen is given in Table 6.1 along with a summary of the shear interface details and interface reinforcement of all eleven specimens.

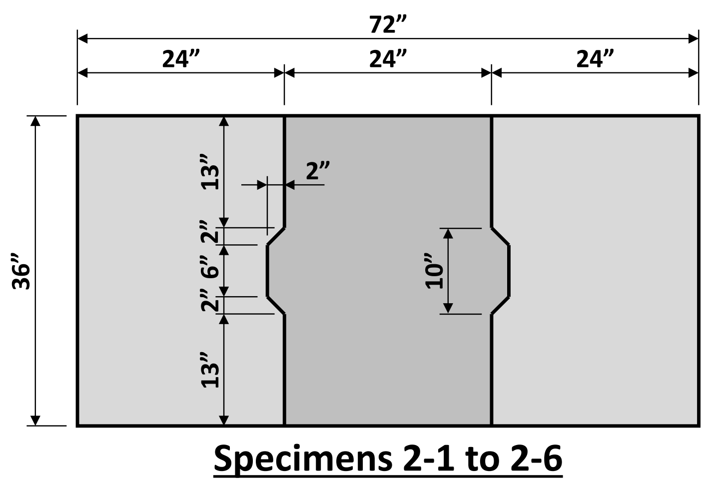


Figure 6.6: Set 2 shear interface detail

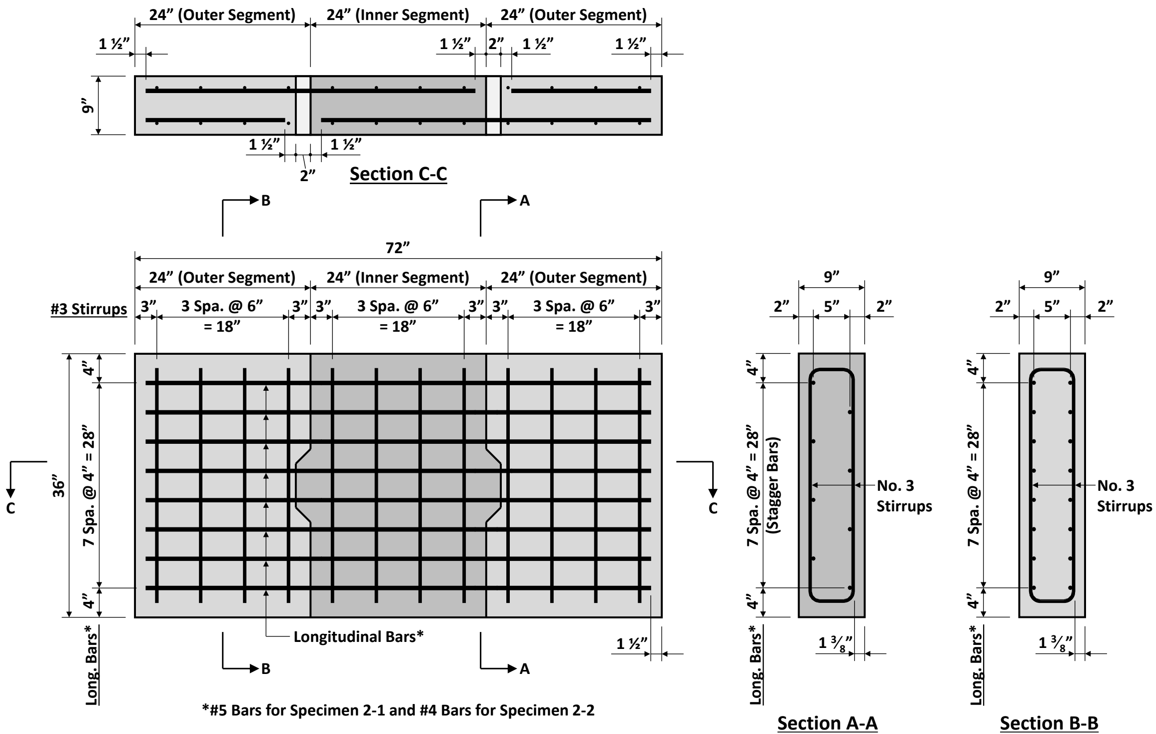


Figure 6.7: Reinforcement details of Specimens 2-1 and 2-2

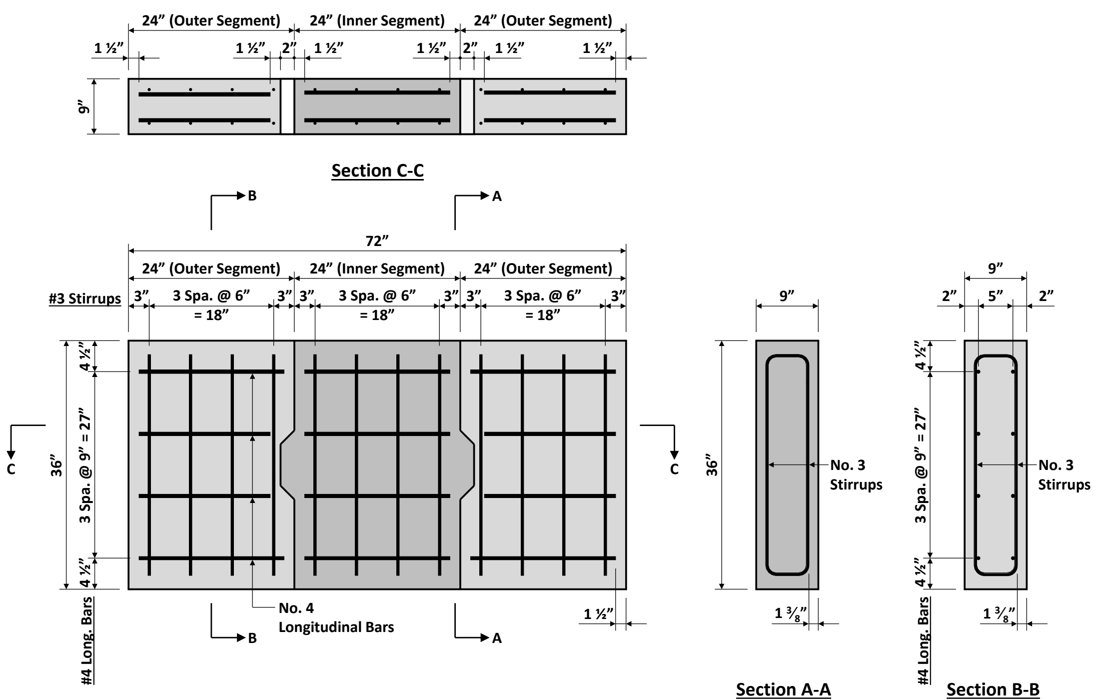


Figure 6.8: Reinforcement details of Specimen 2-3

Table 6.1: Summary of Interface Details, Post-Tensioning Force, and Interface Reinforcement of Push-Through Specimens

	Specimen	Interface Type	Post-Tensioning Force, P_c (kip)	Interface Reinforcement	
				Description	Reinf. Ratio, ρ*
Set 1	**1-1**	Smooth	163	4 - No. 4 Bars	0.0025
	1-2	Single 12-in. Shear Key	162		
	1-3	Two 6-in. Shear Keys	163		
	1-4	1-in. Saw Teeth	163		
	1-5	2-in. Saw Teeth	164		
Set 2	**2-1**	Single 10-in. Shear Key	11	8 - No. 5 Bars	0.0077
	2-2		64	8 - No. 4 Bars	0.0049
	2-3		159	None	0
	2-4		12	4 - No. 4 Bars	0.0025
	2-5		66		
	2-6		202		

*ρ = area of interface reinforcement divided by the nominal cross-sectional area of the specimen

6.2.3 Concrete Mixture for Push-Through Specimens

The concrete mixture design for the outer segments of the Set 1 specimens is presented in Table 6.2. This design was slightly adjusted for all subsequent casts to reduce slump and create a more cohesive concrete mixture. The updated mixture design used for the Set 2 specimens as well as the inner segments of the Set 1 specimens is provided in Table 6.3. This mixture corresponds to the concrete used for the CIP splice regions of the spliced girder test specimens (refer to Section 4.11.1). Both mixtures presented in Tables 6.2 and 6.3 have 700 lbs of cementitious material per cubic yard of concrete and contain 1-in. (TxDOT Grade 4) river gravel as the coarse aggregate.

Table 6.2: Push-through Specimen Concrete Mixture Design – Outer Segments of Set 1

Material	Details	Design Quantity	Units
Cementitious Material	Type I/II Cement	525	lb/yd^3 concrete
	Class F Fly Ash	175	
Coarse Aggregate	River Gravel (1" Nominal)	1,880	
Fine Aggregate	Sand	1,197	
Water	---	242	
Admixtures	High-Range Water Reducer	6.5	oz/cwt
	Water Reducer/Retarder	2.0	

Table 6.3: Push-through Specimen Concrete Mixture Design – Set 2 and Inner Segments of Set 1

Material	Details	Design Quantity	Units
Cementitious Material	Type I/II Cement	525	lb/yd^3 concrete
	Class F Fly Ash	175	
Coarse Aggregate	River Gravel (1" Nominal)	1,880	
Fine Aggregate	Sand	1,221	
Water	---	233	
Admixtures	High-Range Water Reducer	5.5	oz/cwt
	Water Reducer/Retarder	2.0	

6.2.4 Test Setup and Instrumentation

The load frame used for the shear-friction experimental program is shown in Figures 6.9 and 6.10. Load was applied to the inner segment of the test specimens by a vertically oriented hydraulic cylinder, resulting in the development of shear stresses along the interfaces between the inner and outer segments. Steel roller supports located above the outer segments of the test specimens transferred forces to a reaction beam and two transfer girders (refer to Figures 6.9 and 6.10). Loads were ultimately resisted by two sets of four 3-in. diameter steel rods that were threaded into the steel strong floor. A total of eight hollow load cells, each with a capacity of 500 kips, were positioned between the transfer girders and reaction nuts that were threaded onto each of the eight rods. Thus, four load cells were located at both the north and south ends of the test frame, as indicated in Figure 6.9. The sum of the measurements from the load cells was equal to the total force applied to the specimens by the hydraulic cylinder.

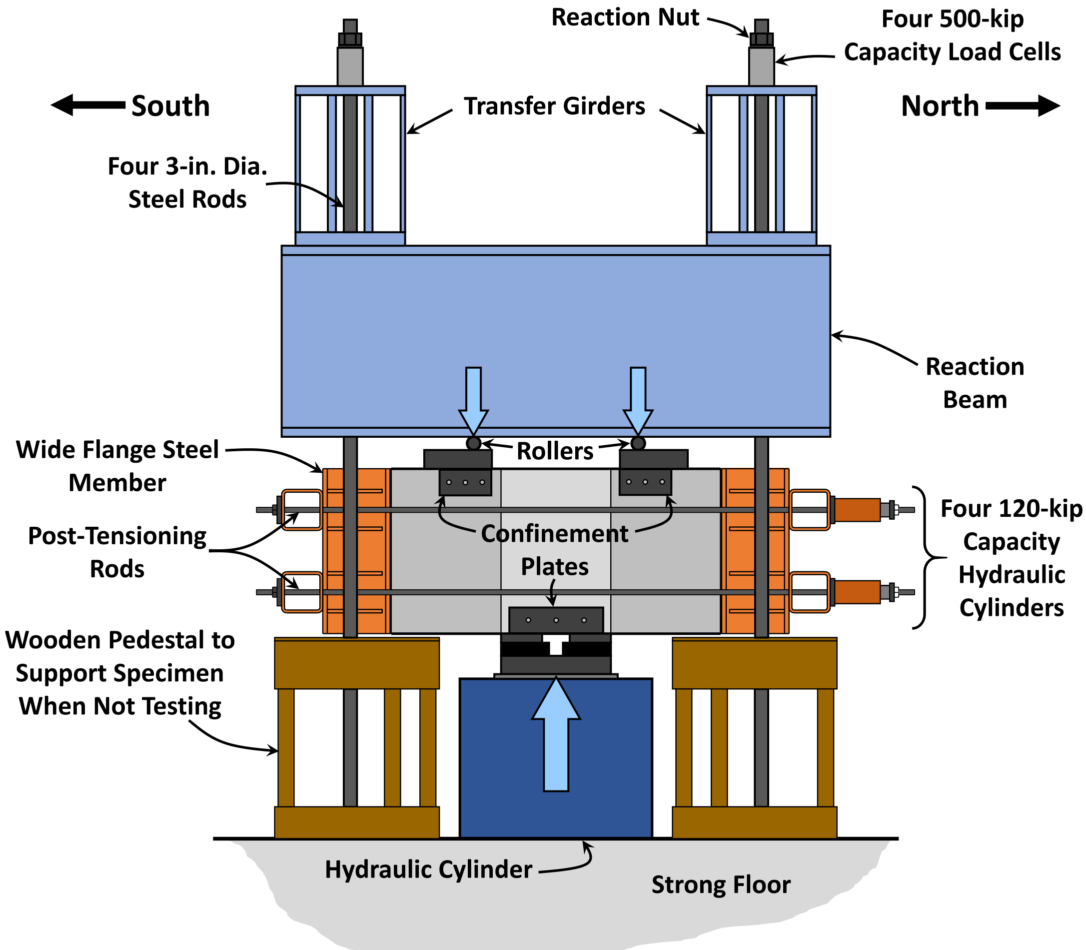


Figure 6.9: Elevation view of test setup for the shear-friction experimental program

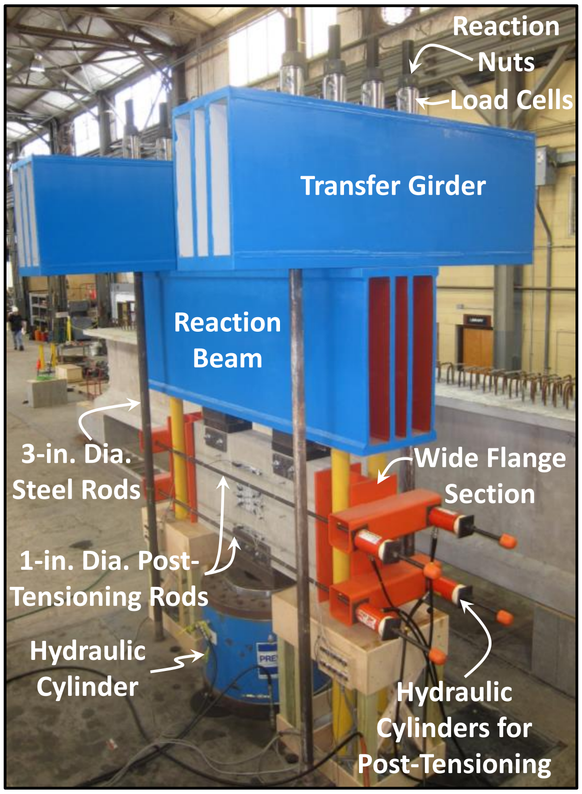


Figure 6.10: Photograph of test setup for the shear-friction experimental program

Steel plates with a thickness of 1 in. were installed on the side faces of the test specimens near the load and supports points to provide confinement and preclude localized concrete crushing (refer to Figures 6.9 and 6.11). Threaded rods extending through the thickness of the specimens were used to clamp the plates to the surface of the concrete.

The specimens were post-tensioned by using four 120-kip capacity hydraulic cylinders. Each cylinder applied tension to a 1-in. diameter steel rod. Compressive forces were transferred to the specimens through wide flange steel members. Pressure transducers were installed in-line with the hydraulic system and were used to ensure the desired post-tensioning force was applied to the specimens. A hydraulic load maintainer was used throughout each test to produce a constant applied force.

The test specimens were instrumented with linear potentiometers to monitor the relative displacements between the inner and outer segments. The linear potentiometers used to measure relative vertical movement are shown in Figure 6.11. The sensors were mounted to the outer segments and measured the relative displacement of an aluminum strip attached at the center of the inner segment. Data gathered from these linear potentiometers are presented and discussed in Sections 6.3.4 and 6.3.5. It should be noted that a linear potentiometer did not function properly for three of the eleven tests. Displacement data corresponding to these sensors are therefore not provided.

Additional information regarding the load frame, post-tensioning system, and instrumentation is provided in Massey (2014).

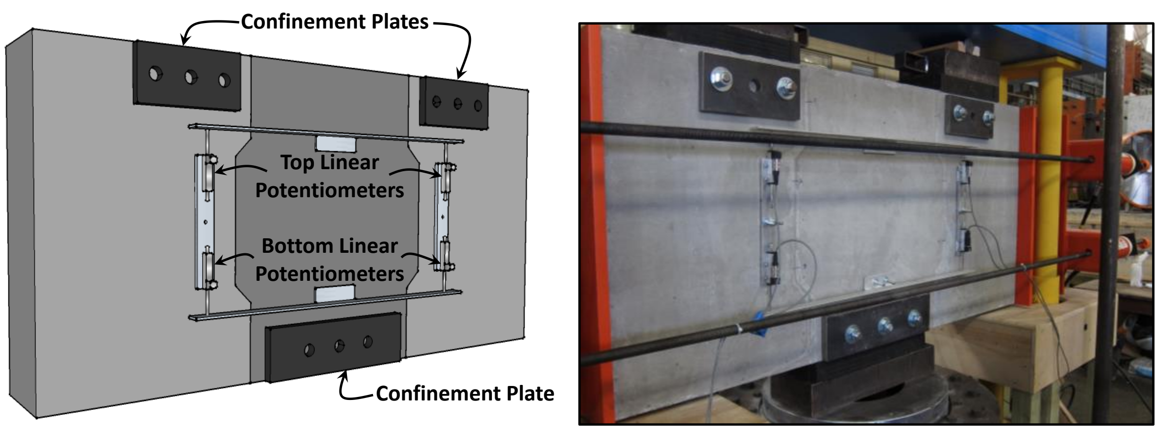


Figure 6.11: Linear potentiometers measuring relative vertical displacements

6.2.5 Test Procedure

For each load test, the specimen was first post-tensioned within the test setup before any vertical load (i.e., shear force) was applied. After confirming that the desired level of axial compression was acting on the specimen, the inner segment was loaded using the vertically oriented hydraulic cylinder, imposing shear stresses on the interfaces between the inner and outer segments. The vertical load was increased monotonically until the specimen failed in shear. The total load was typically applied at increments of approximately 50 or 100 kips in order to monitor the formation and growth of cracks and note any signs of distress. Shear failure was typically indicated by a significant drop in the load resisted by the specimen and notable visual distress at the failed interface.

6.2.6 Quantitative Test Specimen Details

A summary of the quantitative test specimen details for the eleven push-through specimens examined in this chapter are provided in Table 6.4. The variables used in the table are defined below. For strength calculations, the actual width of each specimen, b_{vi}, was used. The width was measured with a caliper at the outer segment corresponding with the failure interface. (The identification of the failure interface is described in Section 6.3.2.) The values of b_{vi} were obtained by averaging the widths measured near both the top and bottom of each specimen. It should also be noted that the yield strengths, f_y, of the mild interface reinforcement provided in Table 6.4 were determined through material tests, and the measured values were used in strength calculations.

A_{vf} = Area of interface shear reinforcement crossing between the inner and outer segments of the specimen (in.^{2})

b_{vi} = Measured width of the specimen (in.)

f'_c = Compressive strength of concrete at the time of testing (ksi)

f'_t = Splitting tensile strength of concrete at the time of testing (ksi)

f_y = Measured yield strength of interface shear reinforcement (ksi)

P_c = Total compressive force applied to the specimen through post-tensioning (kip)

Table 6.4: Summary of Push-through Specimen Details

<table>
<tr><th rowspan="2"></th><th rowspan="2">Specimen</th><th rowspan="2">Interface Type*</th><th rowspan="2">P_c (kip)</th><th colspan="2">Interface Reinforcement</th><th colspan="2">Outer Segment</th><th colspan="2">Inner Segment</th><th rowspan="2">b_{vi} (in.)</th></tr>
<tr><th>A_{vf} (in.^{2})</th><th>f_y (ksi)</th><th>f'_c (ksi)</th><th>f'_t (ksi)</th><th>f'_c (ksi)</th><th>f'_t (ksi)</th></tr>
<tr><td rowspan="5">Set 1</td><td>1-1</td><td>Smooth</td><td>163</td><td>0.80</td><td>65.7</td><td>9.56</td><td>0.84</td><td>8.92</td><td>0.88</td><td>9.18</td></tr>
<tr><td>1-2</td><td>Single 12” SK</td><td>162</td><td>0.80</td><td>65.7</td><td>9.29</td><td>0.81</td><td>9.53</td><td>0.82</td><td>9.07</td></tr>
<tr><td>1-3</td><td>Two 6” SK</td><td>163</td><td>0.80</td><td>65.7</td><td>9.52</td><td>0.82</td><td>9.68</td><td>0.91</td><td>9.10</td></tr>
<tr><td>1-4</td><td>1” ST</td><td>163</td><td>0.80</td><td>65.7</td><td>8.56</td><td>0.88</td><td>9.32</td><td>0.83</td><td>9.01</td></tr>
<tr><td>1-5</td><td>2” ST</td><td>164</td><td>0.80</td><td>65.7</td><td>9.27</td><td>0.84</td><td>9.91</td><td>0.76</td><td>9.02</td></tr>
<tr><td rowspan="6">Set 2</td><td>2-1</td><td rowspan="6">Single 10” SK</td><td>11</td><td>2.48</td><td>72.0</td><td>7.26</td><td>0.74</td><td>6.93</td><td>0.70</td><td>9.21</td></tr>
<tr><td>2-2</td><td>64</td><td>1.60</td><td>61.7</td><td>7.26</td><td>0.74</td><td>6.93</td><td>0.70</td><td>9.17</td></tr>
<tr><td>2-3</td><td>159</td><td>0</td><td>N/A</td><td>7.48</td><td>0.71</td><td>7.48</td><td>0.76</td><td>9.16</td></tr>
<tr><td>2-4</td><td>12</td><td>0.80</td><td>61.7</td><td>7.29</td><td>0.70</td><td>7.59</td><td>0.66</td><td>9.10</td></tr>
<tr><td>2-5</td><td>66</td><td>0.80</td><td>61.7</td><td>7.29</td><td>0.70</td><td>7.59</td><td>0.66</td><td>9.08</td></tr>
<tr><td>2-6</td><td>202</td><td>0.80</td><td>61.7</td><td>7.29</td><td>0.70</td><td>7.59</td><td>0.66</td><td>9.12</td></tr>
</table>

*SK = Shear Key, ST = Saw Teeth

Fabrication of each set of push-through specimens was accomplished with two concrete casts (one for the outer segments and one for the inner segments). The specimens of Set 1 were fabricated separately from those of Set 2. Therefore, a total of four concrete batches are represented in Table 6.4.

6.3 Results and Observations

The results and observations from the push-through tests are summarized in the following sections. Before presenting the test data, however, the method used to determine the shear force acting along the shear interfaces is described, and the failure criterion used for the data analysis is discussed.

6.3.1 Determination of Shear at Interface

The vertical load applied to the specimens was measured by two sets of four load cells placed near the top of the steel rods at both the north and south ends of the load frame, as described in Section 6.2.4. The sum of the forces measured by each set of load cells is denoted by the variables F_N and F_S in Figure 6.12. Due to inherent imperfections in the load frame and test specimens, the values of F_N and F_S were not equal during the shear-friction tests. Therefore, the values of the reactions from the specimens, represented by V_N and V_S in Figure 6.12, were also not equal. To determine the shear forces acting along the interfaces between the segments of the test specimens, the values of V_N and V_S should be calculated. By treating the set of three steel beams illustrated in Figure 6.12 as a simply supported system, the values of V_N and V_S were determined from the measured forces represented by F_N and F_S. The distance between the steel rods of the load frame was approximately 90 in., and the roller supports of the test specimens were typically spaced at 36 in. as shown. A force of 12.6 kips was added to both V_N and V_S to account for the weight of the three steel beams. The resulting values (i.e., V_N + 12.6 kips and V_S + 12.6 kips) were then taken as the shear forces acting along the north and south shear interfaces of the test specimens.

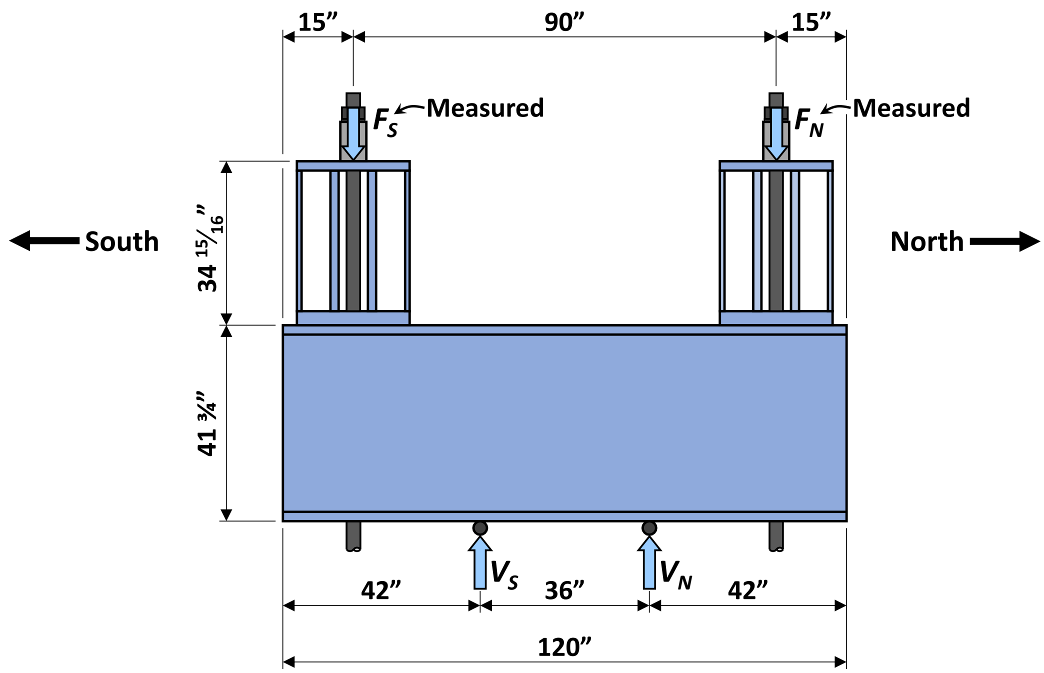
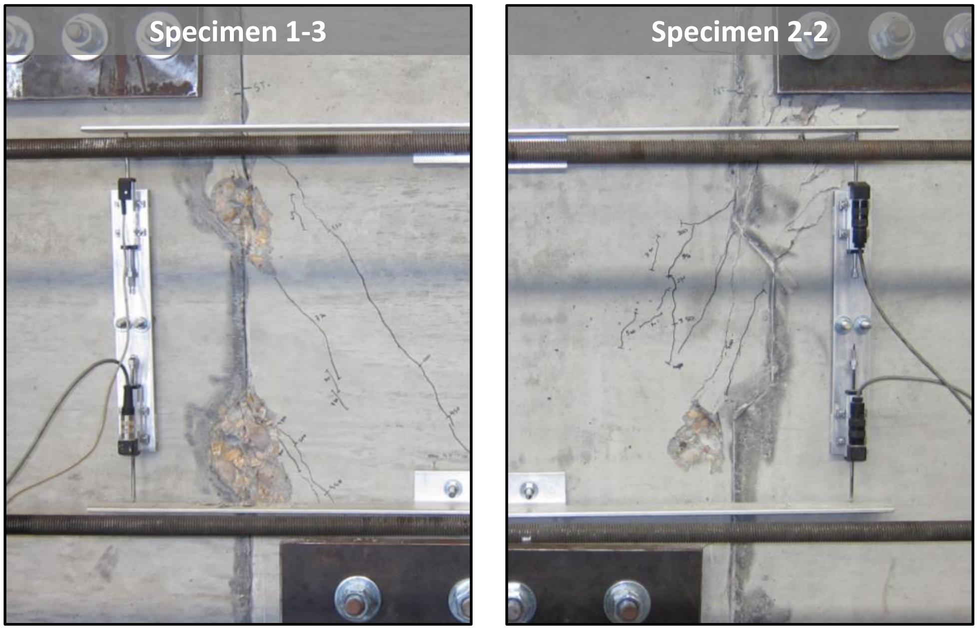


Figure 6.12: Determination of shear forces acting along the shear interfaces of the push-through test specimens

6.3.2 Failure Criterion

The two shear planes (i.e., shear interfaces) of the test specimens both had the potential to exhibit failure. To determine the failure shear force (i.e., ultimate shear force) for each test, the interface that first experienced failure was identified. Failure at an interface was typically characterized by notable visual distress, as shown by Specimens 1-3 and 2-2 in Figure 6.13, along with a significant drop in the shear force resisted at that interface. The failure shear force, or the experimental shear capacity, V_{test}, was defined as the maximum shear force resisted by the interface that first exhibited failure, hereafter called the *failure interface*. It is important to note that, for all eleven specimens, the relative displacements measured by the vertically oriented linear potentiometers (refer to Figure 6.11) across both interfaces were less than 0.08 in. when V_{test} was reached. Due to the small measured displacement values up to the maximum shear force resisted by the failure interface, slip along the interface was not used to define failure of the specimens.



(a) *(b)*

Figure 6.13: Failed interfaces – (a) Specimen 1-3; (b) Specimen 2-2

6.3.3 Summary of Strength Data

A summary of the experimental shear capacities, V_{test}, of the eleven push-through test specimens is provided in Table 6.5. The primary experimental variables are also presented along with the displacement across the failure interface measured by the top linear potentiometer (refer to Figure 6.11) when V_{test} was reached. For Specimen 2-1, the top linear potentiometer did not function properly. The displacement measured by the bottom linear potentiometer is therefore reported for this case. For all other specimens, the top potentiometer measured a larger displacement across the failure interface than did the bottom potentiometer at the maximum shear force. For comparison of the behaviors of the test specimens, data from the top linear potentiometers are consistently used within the following sections. The displacement data from all four linear potentiometers are plotted in Appendix F. It should be noted that the sensors likely measured displacements due to the cracking/crushing of concrete and other deformations in addition to pure slip along the failure interfaces.

Table 6.5: Summary of Strength Data from Push-through Tests

	Specimen	Interface Type	P_c (kip)	A_{vf} (in.^{2})	V_{test} (kip)	Displacement at Failure (in.)
Set 1	**1-1**	Smooth	163	0.80	240	0.034
	1-2	Single 12-in. Shear Key	162	0.80	349	0.041
	1-3	Two 6-in. Shear Keys	163	0.80	408	0.037
	1-4	1-in. Saw Teeth	163	0.80	309	0.029
	1-5	2-in. Saw Teeth	164	0.80	339	0.026
Set 2	**2-1**	Single 10-in. Shear Key	11	2.48	262	0.038*
	2-2		64	1.60	275	0.044
	2-3		159	0	258	0.006
	2-4		12	0.80	129	0.028
	2-5		66	0.80	215	0.036
	2-6		202	0.80	320	0.017

*Data from bottom linear potentiometer

6.3.4 Effect of Shear Interface Details

The five specimens included in Set 1 were tested to study the effect of various shear interface details on interface shear strength and behavior. Specimen 1-1 was fabricated with a smooth interface between the inner and outer segments, while the other four specimens were detailed with some form of indentation (i.e., shear key(s) or saw teeth), as shown in Figure 6.4. Load-displacement plots for the Set 1 specimens are provided in Figure 6.14. In this figure, the shear force acting along the failure interface is plotted against the corresponding displacement values measured by the top linear potentiometer. The experimental shear capacity, V_{test}, for each specimen corresponds with the maximum force reached in the load-displacement plots.

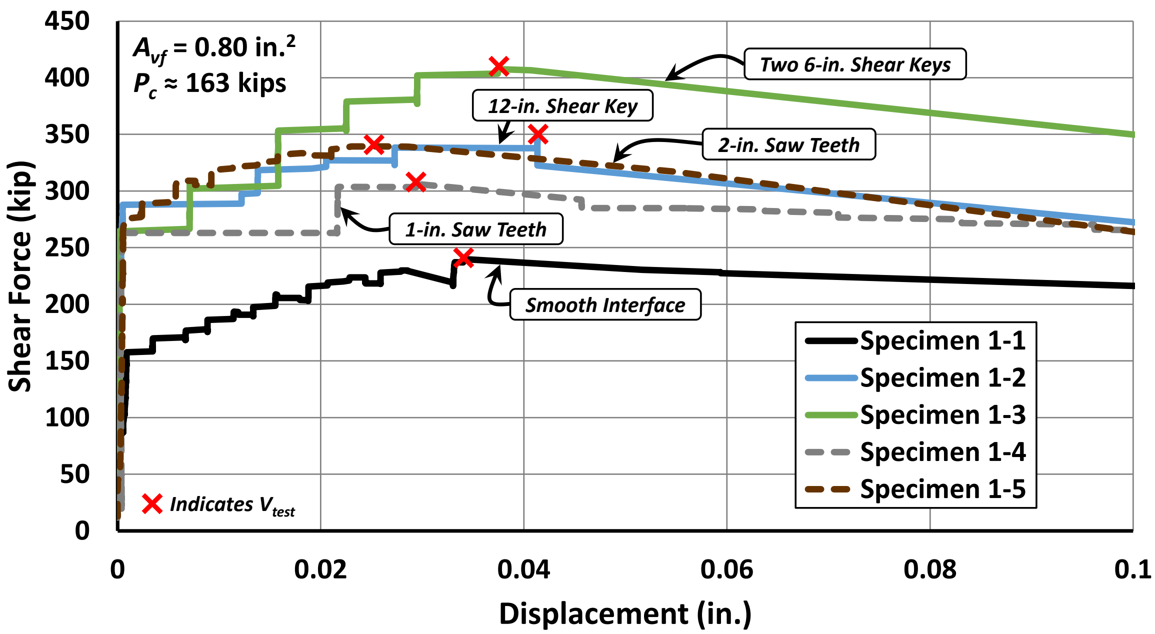


Figure 6.14: Displacement across failure interface of Set 1 specimens measured by the top linear potentiometer

The plots in Figure 6.14 reveal a notable trend in the initial displacements measured at the failure interfaces. According to the data, all the specimens experienced very little relative movement between the segments until a sudden increase in displacement was measured. A significant difference in the shear force corresponding with this "initial displacement" exists between the specimen with the smooth interface (Specimen 1-1) and the other four specimens with indented interfaces. The sudden initial displacement of Specimen 1-1 occurred at a shear force of approximately 160 kips. The other specimens, however, experienced sudden initial displacements at shear forces between 260 and 290 kips. For these specimens with indented interfaces, the initial displacements occurred at similar shear forces despite the differences in the interface details. As the shear force increased beyond this point, variations in the behaviors of the specimens became evident, with Specimen 1-3 resisting the highest shear force (V_{test} = 408 kips).

Although the trend in the initial displacements is clear in the plots of the data from the top linear potentiometers, it should be noted that the plots of the other linear potentiometers did not show a distinct trend. Furthermore, as previously noted, the potentiometers likely detected displacements due to other actions in addition to vertical slip along the interfaces.

The shear strengths of the Set 1 specimens are compared in Figure 6.15. For each specimen, the experimental shear capacity, V_{test}, is normalized by the area of the shear interface, $b_{vi}d$, where b_{vi} is the measured width of the specimen (refer to Section 6.2.6) and d is the nominal specimen depth, or 36 in., as was used in the strength calculations presented in Section 6.3.6. The specimens represented in the figure have been arranged based on their experimental capacities. The specimen with the smooth interface (Specimen 1-1) had the lowest strength out of the Set 1 specimens. The smooth interface after failure is shown in Figure 6.16(a). The indented interfaces of the other four specimens led to the development of interlocking action between the shear keys or saw teeth, resulting in increased strengths. The failure interface at the shear keys of Specimen 1-3 presented in Figure 6.13(a) provides evidence of this interlocking action. For Specimens 1-4 and 1-5, saw teeth provided the interlocking resistance at the interface. Cracking within the 1-in. saw teeth of Specimen 1-4 prior to failure is shown in Figure 6.16(b). Comparing the strengths of the specimens with indented interfaces, the two 6-in. shear keys provided the highest capacity. The 2-in. saw teeth and the single 12-in. shear key resulted in specimens with similar strengths (V_{test} of 339 and 349 kips, respectively).

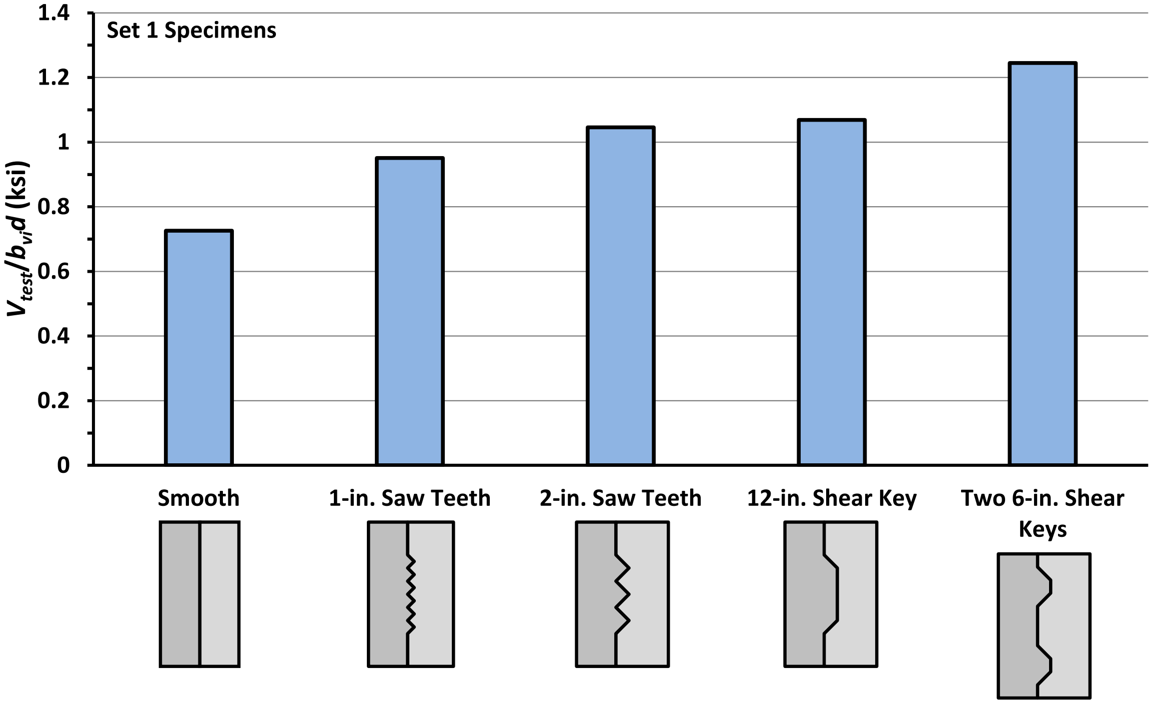
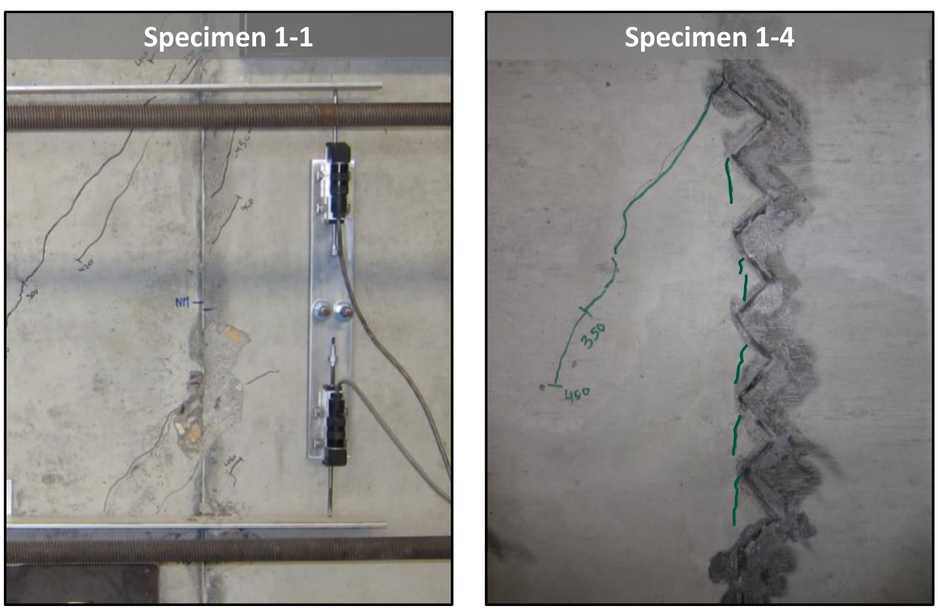


Figure 6.15: Strength comparison of Set 1 specimens



(a) *(b)*

Figure 6.16: Distress at smooth interface and 1-in. saw teeth – (a) Specimen 1-1 after failure; (b) Specimen 1-4 prior to failure

6.3.5 Influence of Interface Reinforcement and Post-Tensioning Force

The primary experimental variables of the six specimens included in Set 2 were the applied post-tensioning force and the area of reinforcement crossing the interfaces between the inner and outer segments. The shear interfaces of all specimens were detailed with a single 10-in. long shear key (see Figure 6.6). The force in the interface reinforcement at yield, $A_{vf}f_y$, and the compressive force, P_c, applied to each Set 2 specimen through post-tensioning are summarized in Table 6.6. Specimen 1-2 had a shear interface detail (12-in. long shear key) similar to that of the Set 2 specimens. It is therefore included in the table and in the following discussion. The specimens in Table 6.6 have been arranged in ascending order based on the sum $A_{vf}f_y + P_c$.

Table 6.6: Summary of Interface Reinforcement ($A_{vf}f_y$), Post-Tensioning Force, and Experimental Capacities of Set 2 Specimens and Specimen 1-2

Specimen	$A_{vf}f_y$ (kip)	P_c (kip)	$A_{vf}f_y + P_c$ (kip)	V_{test} (kip)	$\frac{V_{test}}{b_{vi}d}$ (ksi)
2-4	49.4	12	61	129	0.39
2-5	49.4	66	115	215	0.66
2-3	0	159	159	258	0.78
2-2	98.7	64	162	275	0.83
2-1	178.6	11	189	262	0.79
1-2	52.6	162	214	349	1.07
2-6	49.4	202	251	320	0.98

The load-displacement plots for the Set 2 specimens are presented in Figure 6.17. The plots again show the shear force acting along the failure interface versus the corresponding displacement measured by the top linear potentiometer. (The bottom linear potentiometer was used for Specimen 2-1 due to inaccurate readings produced by the top potentiometer.)

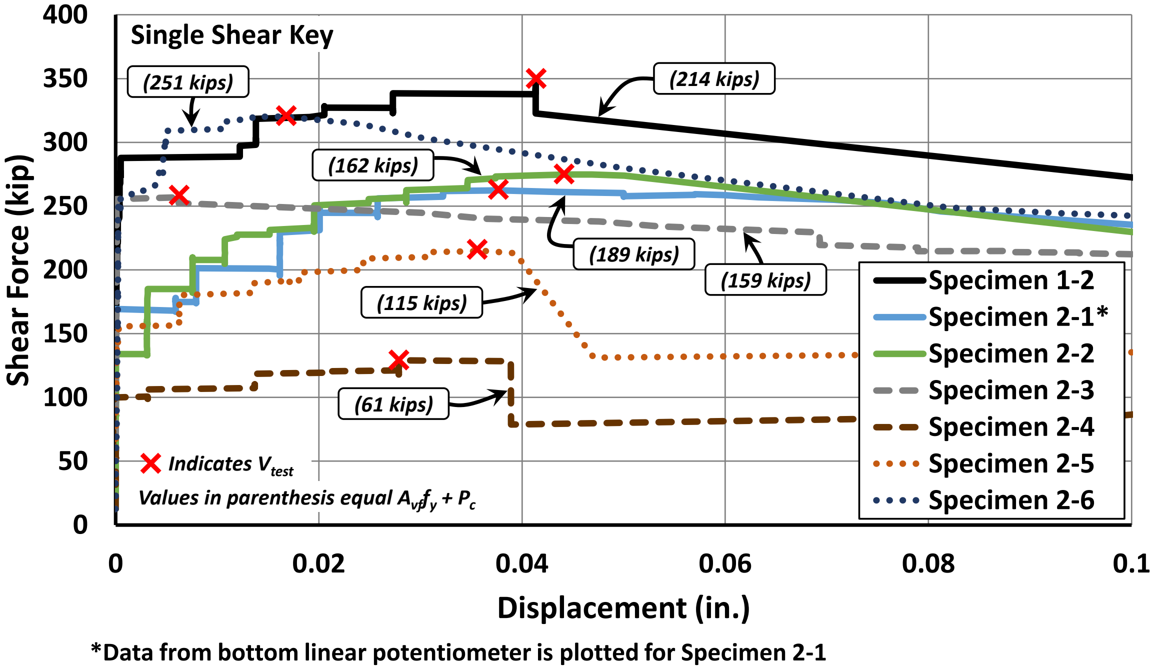


Figure 6.17: Displacement across failure interface of specimens with a single shear key measured by the top* linear potentiometer

Similar to the Set 1 specimens, the displacement data reveal that the Set 2 specimens experienced very little relative movement at the interface until a sudden “initial displacement” was detected. However, unlike the plots for the four Set 1 specimens with indented interfaces (see Figure 6.14), the shear force at which the initial displacement occurred varied significantly for the Set 2 specimens. This trend seems to indicate a relationship between the value of the expression $A_{vf}f_y + P_c$, which was held as a constant for the Set 1 tests, and the shear force corresponding with the initial displacement. Specimen 2-4 had the smallest value for the expression $A_{vf}f_y + P_c$ and experienced an initial displacement at a shear force of only 101 kips. The specimen also exhibited the lowest shear strength. Specimens 1-2, 2-3, and 2-6 were subjected to the largest compressive forces (162, 159, and 202 kips, respectively) and exhibited initial displacements at higher shear forces than the other specimens according to the data in

Figure 6.17. Despite these observations, the relative influence that can be attributed to the interface reinforcement ($A_{vf}f_y$) and the compressive force (P_c) on the initial displacement is not clear.

Load-displacement plots for all four vertically oriented linear potentiometers are presented in Appendix F. The data reveal that the trends in the initial displacements described above are less evident in the plots for the bottom linear potentiometers.

The normalized experimental shear capacities of the specimens with a single shear key (i.e., Set 2 specimens and Specimen 1-2) are plotted in Figure 6.18 against the expression $A_{vf}f_y + P_c$. As discussed in Section 6.3.6, the nominal shear resistance within the shear-friction provisions of ACI 318-14, AASHTO LRFD (2014), and Eurocode 2 is a function of this expression. The plot in Figure 6.18 reveals a clear trend between the shear strength and the combined effects of the interface reinforcement and the post-tensioning force. The same information is provided in Figure 6.19. The data points, however, have been replaced by bars that show the individual contributions of the interface steel ($A_{vf}f_y$) and the post-tensioning force (P_c) to the sum $A_{vf}f_y + P_c$. The strength data follow a seemingly linear trend regardless of the relative contributions of the two components. Specimens 2-1, 2-2, and 2-3, each with a relatively similar value for $A_{vf}f_y + P_c$, exhibited comparable strengths despite significant differences between the individual contributions of reinforcement and post-tensioning force. The push-through tests therefore verify the additive nature of these two components to the interface shear strength as assumed in existing shear-friction design provisions.

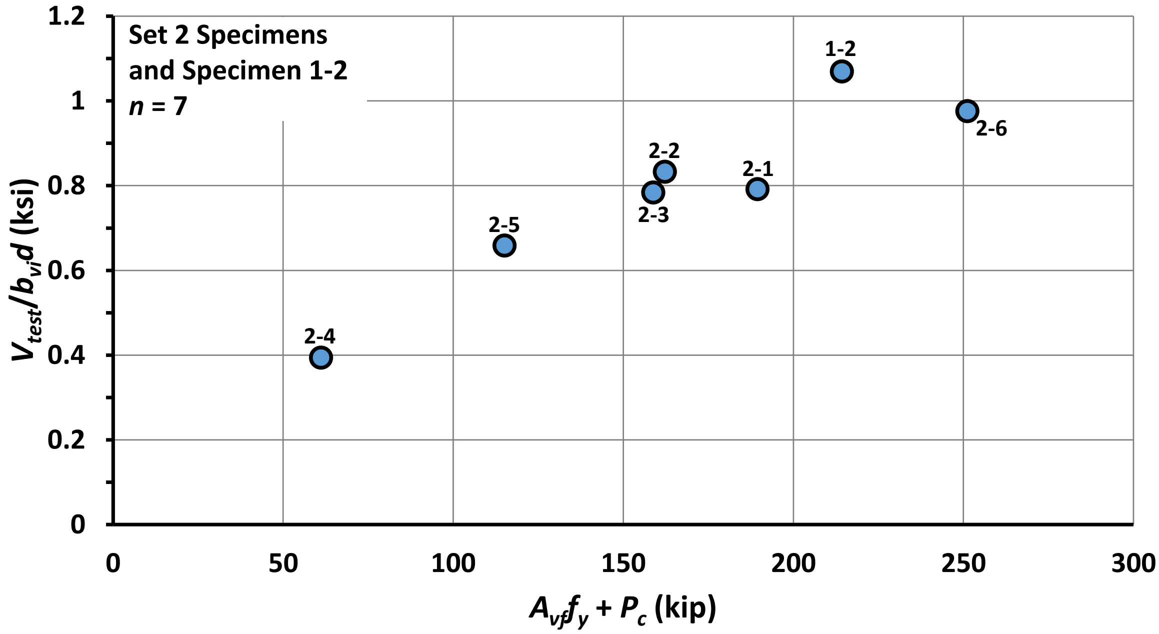


Figure 6.18: Failure shear stress versus the force in the interface reinforcement at yield plus the compressive force acting across the shear plane – simple plot

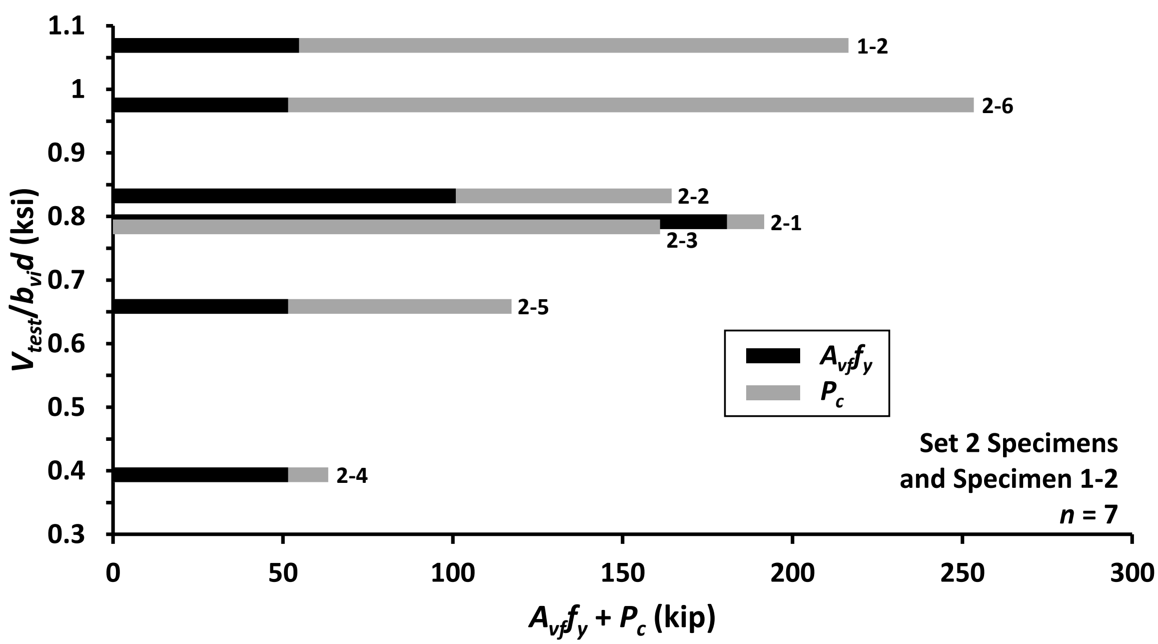


Figure 6.19: Failure shear stress versus the force in the interface reinforcement at yield plus the compressive force acting across the shear plane – detailed plot

6.3.6 Evaluation of Shear-Friction Design Provisions

In this section, the experimental capacities of the eleven push-through test specimens are compared to their calculated strengths based on the shear-friction design provisions of ACI 318-14, AASHTO LRFD (2014), and Eurocode 2. The relevant provisions from each design code are first introduced. The design expressions are then evaluated using the results of the push-through tests.

6.3.6.1 ACI 318-14 Provisions

According to Section 22.9.4 of ACI 318-14, the nominal shear strength, V_n, is calculated as follows when the shear-friction reinforcement is provided perpendicular to the shear plane:

$$V_n = A_{vf} f_y \mu \tag{6.1}$$

where:

A_{vf} = Area of shear-friction reinforcement (in.^{2})

f_y = Specified yield strength of reinforcement (psi)

V_n = Nominal interface shear strength (lb)

μ = Coefficient of friction

The provisions state that permanent net compression acting across the shear interface can be treated as additive to the contribution of interface steel, $A_{vf}f_y$. The value of the coefficient of friction, μ, is determined based on the details of the surface at the shear plane. Table 22.9.4.2 of ACI 318-14 specifies that the value of μ be taken as 1.0λ for a shear surface "intentionally roughened to a full amplitude of approximately ¼ in." For a shear surface that is not intentionally roughened, the value is equal to 0.6λ. In both cases, the factor λ is equal to 1.0 for normal-weight concrete. For purposes of evaluating the code expressions, the shear interfaces of the ten push-through specimens of the experimental program with shear keys or saw teeth were considered to be intentionally roughened. The factors for a surface that is not intentionally roughened were used to evaluate the failure interface of Specimen 1-1 which had no shear keys or saw teeth.

6.3.6.2 AASHTO LRFD (2014) Provisions

Unlike the shear-friction provisions of ACI 318-14, both the AASHTO LRFD (2014) and Eurocode 2 provisions include a "cohesion term" that is added to the contribution of the interface reinforcement and the compressive force acting normal to the shear plane. The commentary of the AASHTO LRFD (2014) specifications explains that the term considers the influence of cohesion and/or aggregate interlock on the shear resistance. According to Article 5.8.4.1 of AASHTO LRFD (2014), the nominal shear resistance, V_{ni}, is determined from the following expression:

$$V_{ni} = cA_{cv} + \mu\left(A_{vf} f_y + P_c\right) \tag{6.2}$$

where:

A_{cv} = Area of concrete resisting interface shear transfer (in.^{2})

A_{vf} = Area of shear-friction reinforcement within area A_{cv} (in.^{2})

c = Cohesion factor (ksi)

f_y = Specified yield strength of reinforcement (ksi)

P_c = Permanent net compressive force acting normal to the shear plane (kip)

V_{ni} = Nominal interface shear resistance (kip)

μ = Coefficient of friction

The values of c and μ both depend on the details of the shear plane. For normal-weight concrete with a surface intentionally roughened to an amplitude of ¼ in., the values of c and μ are 0.24 ksi and 1.0, respectively. For a surface not intentionally roughened, the values of c and μ are 0.075 ksi and 0.6, respectively. Similar to the treatment of the ACI 318-14 provisions, the AASHTO LRFD (2014) factors for a surface not intentionally roughened were used to evaluate Specimen 1-1, while all other specimens were evaluated using the factors for an intentionally roughened surface.

6.3.6.3 Eurocode 2 Provisions

According to Section 6.2.5 of Eurocode 2, the design interface shear resistance, v_{Rdi}, given in terms of stress is expressed as follows:

$$v_{Rdi} = cf_{ctd} + \mu\sigma_n + \rho f_{yd}(\mu \sin\alpha + \cos\alpha) \leq 0.5\nu f_{cd} \qquad \textit{(6.3)}$$

where:

c = Factor that depends on the roughness of the shear interface

f_{cd} = Design value of compressive strength of concrete (MPa)

f_{ctd} = Design value of direct tensile strength of concrete (MPa)

f_{yd} = Design value of yield strength of reinforcement (MPa)

v_{Rdi} = Design interface shear resistance (MPa)

α = Angle between the shear-friction reinforcement and the shear interface, not to be taken as less than 45° or greater than 90° (degrees)

μ = Factor that depends on the roughness of the shear interface

ν = Strength reduction factor (taken as 1.0 for comparisons of calculated strengths to test results)

ρ = Area of shear-friction reinforcement adequately developed on both sides of the interface divided by the area of the joint

σ_n = Stress resulting from the minimum external force acting normal to the shear interface (MPa)

The value of the normal stress, σ_n, must be less than $0.6f_{cd}$ if compressive. The provisions also state that the term cf_{ctd} should be taken as zero when the normal stress is tensile. It should be noted that AASHTO LRFD (2014) does not include such a stipulation for the "cohesion term" when the force, P_c, is tensile. Furthermore, the cohesion term within Eurocode 2 is directly dependent on the tensile strength of the concrete, unlike the corresponding term in AASHTO LRFD.

Section 6.2.5 of Eurocode 2 includes values of c and μ for very smooth, smooth, rough, and indented surfaces. To use the factors for an indented surface, specific geometrical requirements for the indentations must be met. Although the shear keys and saw teeth of the push-through specimens do not comply with every geometrical requirement, it was determined that the interfaces of the specimens best match the description of an indented surface compared to the descriptions of the other surface details (i.e., very smooth, smooth, and rough). The values of c and μ for an indented surface are 0.50 and 0.9, respectively. The interface of Specimen 1-1 with no indentations was determined to best match the qualifications to classify it as a very smooth surface. The value of c for a very smooth surface is specified to range from 0.025 to 0.10, while the value of μ is equal to 0.5. For Specimen 1-1, a value of 0.10 was assumed for the c-factor considering that the outer segments were cast in wooden forms, resulting in a rougher surface than would be produced if steel or smooth plastic forms were used.

To obtain the nominal shear strength of the test specimens in units of force, the shear resistance calculated using Equation 6.3 was multiplied by the interface area, or

$b_{vi}d$. Within Equation 6.3, the design values f_{cd} and f_{yd} were replaced with the measured strengths of the concrete and reinforcement, respectively. The design direct tensile strength of concrete, f_{ctd}, was substituted with the average direct tensile strength, f_{ctm}. The values of f_{ctm} were estimated using the relationships provided in Table 3.1 of Eurocode 2, modified as follows to include the measured concrete compressive strength, f'_c:

$$f_{ctm} = 0.30 f'_c{}^{(2/3)} \text{ for } f'_c \leq 50 \text{ MPa (7.25 ksi)}$$

$$f_{ctm} = 2.12 \cdot \ln\left[1 + {(f'_c + 8 \text{ MPa})}/{10}\right] \text{ for } f'_c > 50 \text{ MPa (7.25 ksi)} \tag{6.4}$$

where f_{ctm} and f'_c have units of MPa. The lesser of the values of f'_c for both the inner and outer segments of the test specimens were used in strength calculations.

6.3.6.4 Comparison of Tested Capacities to Calculated Strengths

The shear-friction design provisions in ACI 318-14, AASHTO LRFD (2014), and Eurocode 2 were evaluated by comparing the calculated strengths of the push-through test specimens to their experimental shear capacities. The values of the c- and μ-factors used to calculate the nominal shear resistance of each test specimen are summarized in Table 6.7 for the three design codes. Both ACI 318-14 and AASHTO LRFD (2014) give factors to use within the shear-friction design expressions for surfaces that are considered to be "intentionally roughened" to an amplitude of ¼ in. or "not intentionally roughened." Since qualifying examples of intentionally roughened surfaces are not provided, the shear keys and saw teeth of the push-through test specimens may not comply with the requirements to be classified as such a surface. For purposes of evaluating the shear-friction code expressions, however, the interfaces of the specimens with shear keys or saw teeth were considered to be intentionally roughened, as previously discussed. For comparisons with the Eurocode 2 calculated strengths, these interfaces were classified as "indented."

Table 6.7: Values of *c*- and μ-Factors Used within Shear-Friction Expressions

Surface Detail	*c*			μ		
	ACI 318	AASHTO LRFD	Eurocode 2	ACI 318	AASHTO LRFD	Eurocode 2
Not Intentionally Roughened or Very Smooth (Specimen 1-1)	-	0.075 ksi	0.10	0.6	0.6	0.5
Int. Roughened or Indented (Specimens 1-2 to 2-6)	-	0.24 ksi	0.50	1.0	1.0	0.9

The ratio of the experimental capacities, V_{test}, to the calculated shear strengths, V_n, of all eleven push-through test specimens are presented in Figure 6.20 for the three shear-friction code expressions. The experimental and calculated strengths along with the shear strength ratios, V_{test}/V_n, are also provided in Table 6.8. It should be noted that all three specifications impose upper limits on the calculated shear strength, some of which are dependent on the compressive strength of concrete, f'_c. The lesser of the values of f'_c for both the inner and outer segments were used when checking the upper limits for the push-through specimens. None of the limits, however, governed the calculated strengths of the eleven test specimens.

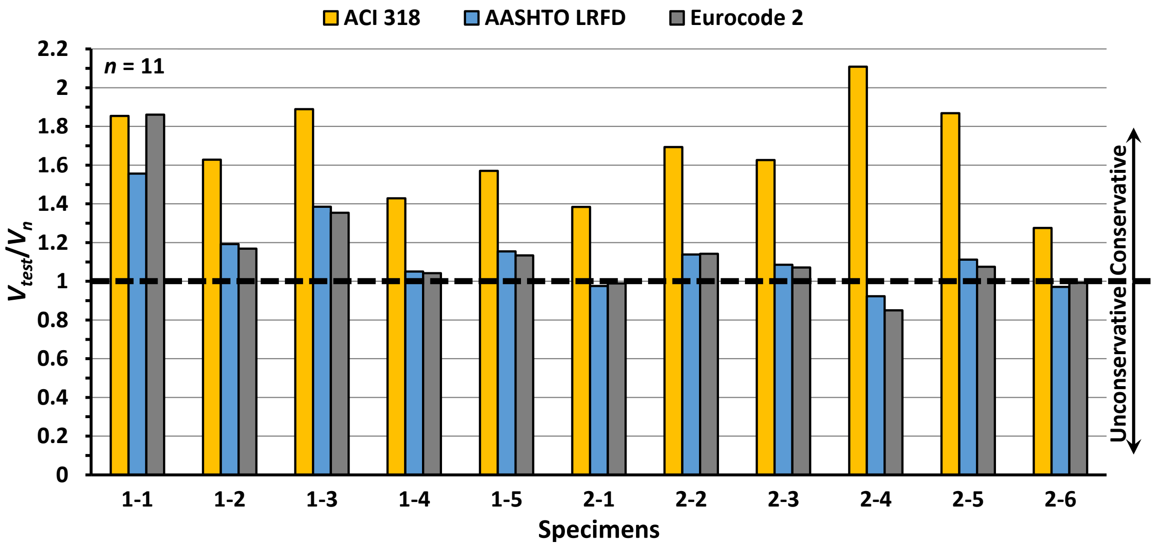


Figure 6.20: Shear strength ratios, V_{test}/V_n, of push-through test specimens considering three shear-friction code expressions

Table 6.8: Summary of Comparisons between Experimental Capacities and Calculated Strengths

Specimen	V_{test} (kip)	V_n (kip)			$\frac{V_{test}}{V_{ACI}}$	$\frac{V_{test}}{V_{AASHTO}}$	$\frac{V_{test}}{V_{EC2}}$
		V_{ACI}	V_{AASHTO}	V_{EC2}			
1-1	240	129	154	129	1.85	1.56	1.86
1-2	349	214	293	298	1.63	1.19	1.17
1-3	408	216	294	301	1.89	1.39	1.35
1-4	309	216	294	296	1.43	1.05	1.04
1-5	339	216	294	299	1.57	1.15	1.13
2-1	262	189	269	265	1.38	0.98	0.99
2-2	275	162	241	241	1.69	1.14	1.14
2-3	258	159	238	241	1.63	1.09	1.07
2-4	129	61	140	152	2.11	0.92	0.85
2-5	215	115	193	200	1.87	1.11	1.08
2-6	320	251	330	323	1.28	0.97	0.99
				Mean:	1.67	1.14	1.15
				Standard Deviation:	0.24	0.18	0.25

Considering Figure 6.20, the ACI 318-14 shear-friction provisions generally result in strength estimates that are significantly more conservative than those of the other two specifications. Although all three codes consider the additive strength contributions of the interface reinforcement and the compressive force acting normal to the shear interface, only AASHTO LRFD (2014) and Eurocode 2 include a "cohesion term" that is specified based on the details of the shear plane. Without a cohesion term, the ACI 318-14 expression tends to provide additional conservatism as shown in Figure 6.20.

The shear predictions using all three code expressions are conservative for the five Set 1 test specimens for which the only experimental variable was the shear interface detail. For the Set 2 specimens, the shear strength ratios, V_{test}/V_n, for the AASHTO LRFD (2014) provisions range from 0.92 to 1.14, while V_{test}/V_n values range from 0.85 to 1.14 for Eurocode 2. The least conservative estimate of both specifications is for Specimen 2-4, which had a relatively small strength contribution from the interface reinforcement and the post-tensioning force.

To further evaluate the strength predictions for the six specimens of Set 2 and Specimen 1-2 (i.e., specimens with a single shear key), the shear strength ratio obtained from each shear-friction design expression is plotted in Figure 6.21 against the value of $A_{vf}f_y + P_c$ for each specimen. Without the inclusion of a cohesion term, the ACI 318-14 shear-friction provisions result in an obvious downward trend as the contribution of interface reinforcement and compressive force increases. The resulting shear strength estimates are rather conservative for some specimens. For example, V_{test}/V_n for Specimen 2-4 is 2.11. The downward trend is eliminated with the addition of the cohesion term within the AASHTO LRFD (2014) and Eurocode 2 shear-friction expressions. The arrangement of the data points in Figure 6.21 resulting from these two provisions exhibits similar patterns, with Specimen 2-4 having the lowest shear strength ratio.

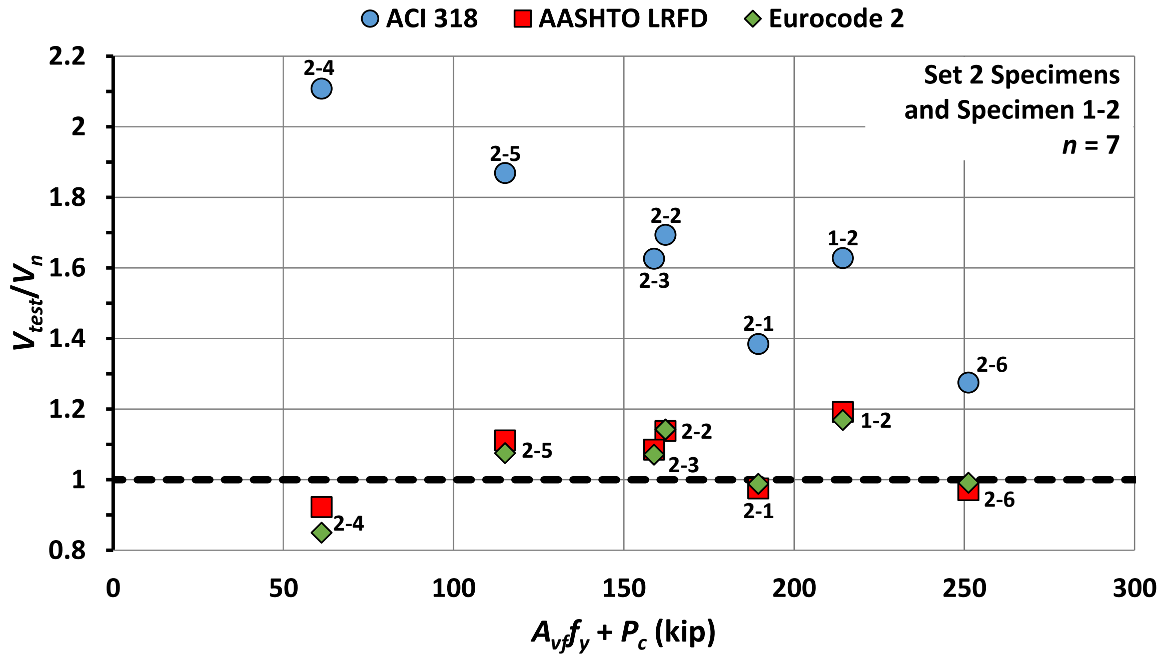


Figure 6.21: Evaluation of Set 2 specimens and Specimen 1-2 using current shear-friction code expressions

The relative contributions of the three components within the shear-friction strength expressions are displayed in Figure 6.22 for each push-through specimen. The experimental capacities are also included in the figure. Considering Specimen 1-1, the *c*- and μ-factors for a surface that is not intentionally roughened (considered "very smooth" in Eurocode 2) result in all three specifications giving conservative strength estimates. The small values for *c* within AASHTO LRFD (2014) and Eurocode 2 for such a surface result in only a minor contribution from the cohesion term. For the remaining specimens of Set 1 (Specimens 1-2 to 1-5), the AASHTO LRFD and Eurocode 2 expressions provide nearly the same calculated strengths. Differences in the experimental capacities were influenced by the variations of the shear interface details.

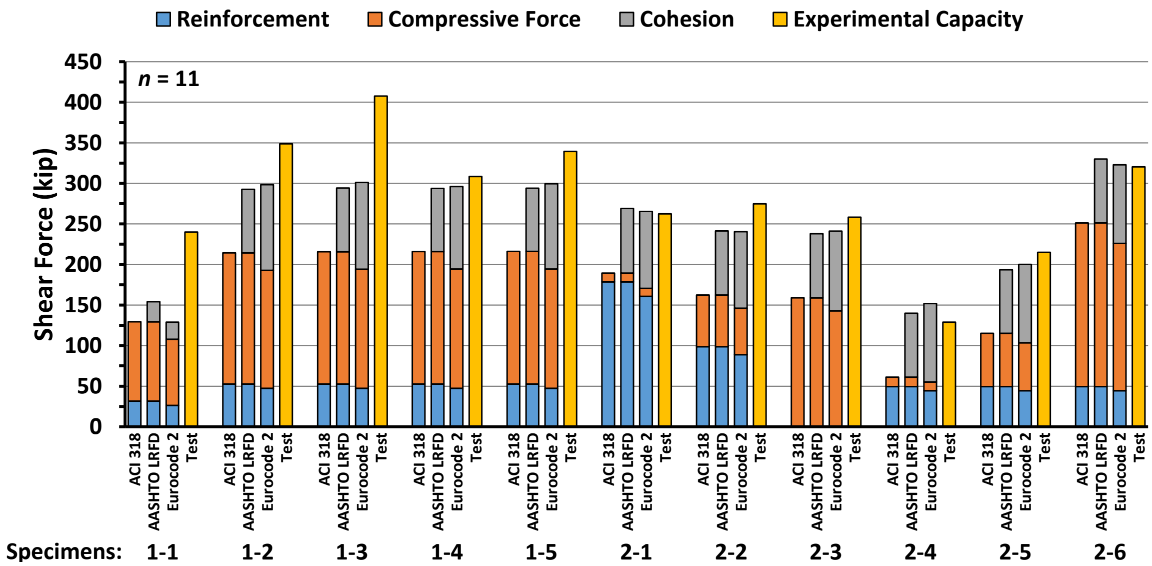


Figure 6.22: Comparisons of calculated strengths and experimental capacities of push-through specimens

Considering the Set 2 specimens, the ACI 318-14 provisions provide conservative estimates in all cases. The AASHTO LRFD (2014) and Eurocode 2 expressions, however, give more accurate results when compared to the calculated capacities. These two expressions result in shear strength ratios greater than 0.95 in all cases except for Specimen 2-4. As illustrated in Figure 6.22, the AASHTO LRFD and Eurocode 2 provisions attribute a relatively significant portion of the predicted capacity of Specimen 2-4 to the cohesion term. Its contribution to the calculated strength is larger than the combined effects of the interface reinforcement and post-tensioning force. The code expressions may have over-predicted the contribution of the cohesion term, resulting in an unconservative strength estimate.

As shown in Figure 6.22, the value of the cohesion term is relatively constant for all the test specimens with indented interfaces. Therefore, as the contribution of the interface reinforcement and compressive force ($A_{vf}f_y + P_c$) decreases, the cohesion term represents a larger percentage of the total calculated strength according to the AASHTO

LRFD (2014) and Eurocode 2 shear-friction expressions. Without the consideration of a cohesion term, as in the ACI 318-14 provisions, the strengths of the Set 2 specimens corresponding with low values for the expression $A_{vf}f_y + P_c$ are estimated with more conservatism than other Set 2 specimens. For this reason, the plot of the shear strength ratios, V_{test}/V_n, resulting from the ACI 318-14 provisions (see Figure 6.21) demonstrates a downward trend. The shear strength ratio is therefore largest for Specimen 2-4. With the inclusion of the cohesion term, however, the shear strength ratios resulting from AASHTO LRFD and Eurocode 2 were smallest for this specimen.

The normalized shear strengths of the ten test specimens with indented interfaces are plotted in Figure 6.23 along with the ACI 318-14 and AASHTO LRFD (2014) shear-friction expressions for surfaces that are intentionally roughened. Please note that the ACI 318-14 and AASHTO LRFD plots were created using the nominal area of the shear interfaces. The overall conservatism of the ACI 318-14 provisions is again demonstrated in the figure, while the AASHTO LRFD expression fits the data fairly well, especially considering the Set 2 specimens. It should be noted that the shear interfaces of the Set 2 specimens were all detailed with a single 10-in. shear key. Therefore, the plot indicates that the AASHTO LRFD expression for a surface that is intentionally roughened predicts the strength of such specimens with reasonable accuracy but results in some unconservative estimates. The scatter among the data points for the Set 1 specimens in Figure 6.23 may imply that the level of conservatism offered by the code expressions depends on the specific details of the indented or roughened surface.

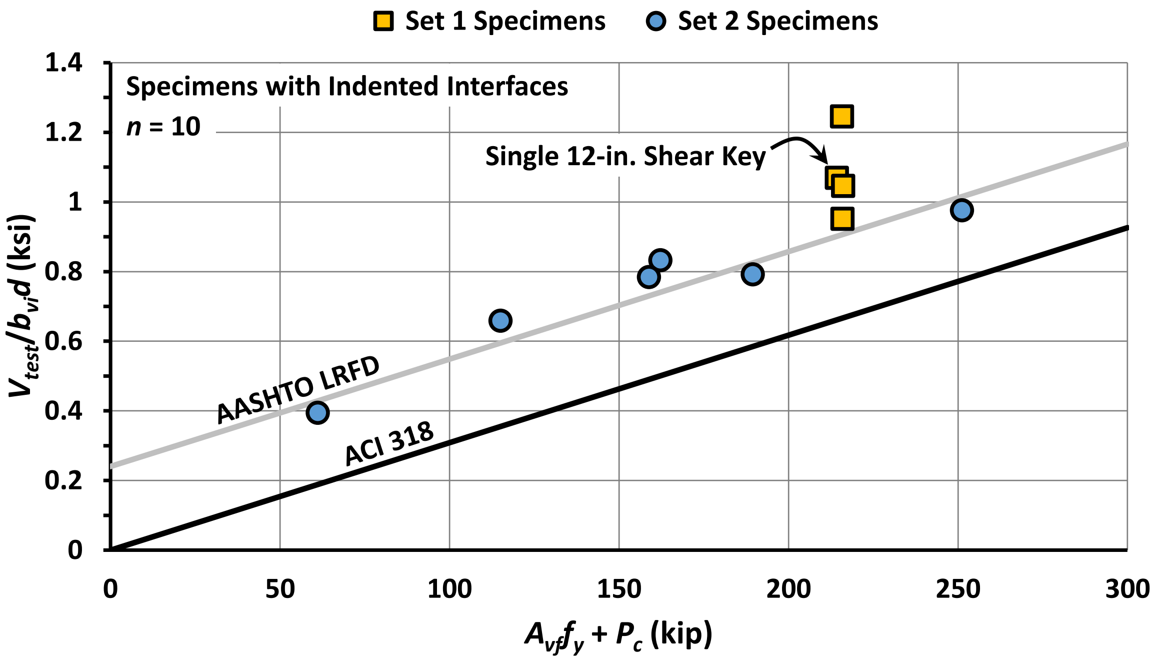


Figure 6.23: Comparisons of experimental capacities and the shear-friction expressions of the ACI 318-14 and AASHTO LRFD (2014) provisions

A summary of the performance of the shear-friction code provisions considering the eleven push-through test specimens is provided in Table 6.9. Within this table, please note that the Eurocode 2 provisions have been evaluated considering two different assumptions for the classification of the surfaces detailed with shear keys or saw teeth (Specimens 1-2 to 2-6). The first assumption applies the *c*- and μ-factors for an "indented" surface, as was assumed for the preceding analysis within this section. The test specimens were then evaluated using the Eurocode 2 shear-friction expression with the *c*- and μ-factors specified for a "rough" surface (c = 0.40 and μ = 0.7).

Table 6.9: Performance of Shear-Friction Design Provisions – Shear Strength Ratio, V_{test}/V_n, Statistics

***n* = 11 tests**	**V_{test}/V_n**					
Design Provision	**Max.**	**Min.**	**Mean**	**% Unconservative***	**Standard Deviation**	**COV****
ACI 318	2.11	1.28	1.67	0%	0.24	0.14
AASHTO LRFD	1.56	0.92	1.14	27.3%	0.18	0.16
Eurocode 2 (Indented)	1.86	0.85	1.15	27.3%	0.25	0.22
Eurocode 2 (Rough)	1.86	1.07	1.42	0%	0.21	0.15

*Unconservative = $V_{test}/V_n < 1.0$
**COV = Coefficient of Variation = Standard Deviation/Mean

Considering the summaries of the four cases presented in Table 6.9, the ACI 318-14 provisions are the most conservative, with a mean shear strength ratio, V_{test}/V_n, of 1.67. Both the AASHTO LRFD (2014) shear-friction expression and the Eurocode 2 specifications assuming indented interfaces result in a lower mean but provide unconservative strength estimates. The Eurocode 2 specifications assuming indented interfaces also have the largest coefficient of variation. Although AASHTO LRFD overestimates the strength of three of the eleven specimens, the values of V_{test}/V_n for these tests are 0.92, 0.97, and 0.98, indicating relatively accurate strength predictions.

Considering the possible classifications for surface details provided in Eurocode 2 (i.e., very smooth, smooth, rough, and indented), it was determined that the interfaces of the push-through test specimens with shear keys or saw teeth best match the description of an indented surface. However, the performance of Eurocode 2 is improved if these interfaces are classified as rough surfaces. This case arguably exhibits the best performance compared to the other three cases in Table 6.9, with no unconservative estimates and an average shear strength ratio of 1.42. Furthermore, the associated variability in the data is similar to that of ACI 318-14 and AASHTO LRFD (2014). A comparison of the shear strength ratios as shown in Figure 6.20 is repeated in Figure 6.24 for Specimens 1-2 to 2-6 with the addition of the Eurocode 2 expression with the *c*- and μ-factors for a "rough" surface.

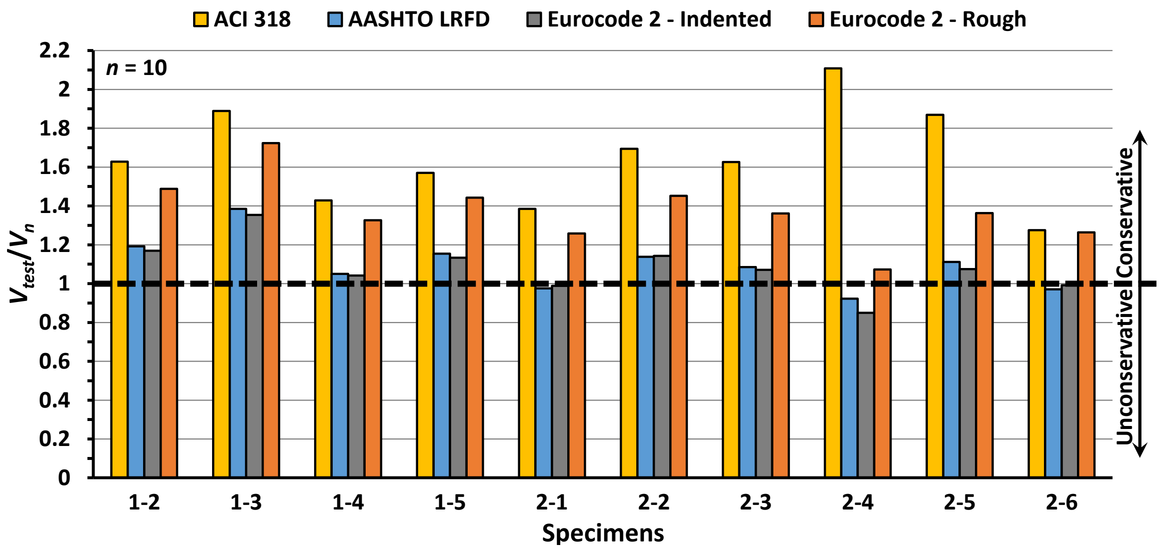


Figure 6.24: Shear strength ratios, V_{test}/V_n, of push-through test specimens including the Eurocode 2 provisions assuming a rough surface

Classifying the interfaces of the specimens detailed with shear keys or saw teeth as "intentionally roughened" (or "rough") surfaces with respect to the code provisions provides reasonable results in most cases. However, further research with extensive testing of shear interfaces that are commonly specified for spliced girders could result in cohesion and friction factors that are calibrated for specific indented surface details. This would improve the performance of shear-friction provisions and eliminate the confusion caused by current specifications when deciding what factors best apply to a surface detailed with indentations.

6.3.7 Application to Spliced Girders

The results of the shear-friction experimental program can be extended to the interfaces between the precast segments and the splice regions of spliced girder bridges. Comparing the strengths of the Set 1 test specimens (refer to Figure 6.15), the interlocking action provided by indented surfaces did result in increased capacities. Considering the four types of indented surfaces that were tested within Set 1, only the specimen with two 6-in. shear keys exhibited a shear strength greater than that of the specimen with the single 12-in. shear key. These observations along with the satisfactory performance of the splice regions of the large-scale spliced girder test specimens help indicate that a simple shear key with an inset of at least 2 in. is suitable for application within field structures.

Considering the code expressions previously discussed, the high post-tensioning force applied to typical spliced girders along with the effect of reinforcement crossing the splice region interface is expected to dominate the calculated shear-friction capacity at splice regions compared to the cohesion term. With a large post-tensioning force applied to the girders, the cohesion term can be conservatively neglected from the shear-friction design expressions without a significant effect on the resulting calculated capacity.

The upper limits imposed on the calculated interface shear strengths specified within ACI 318-14, AASHTO LRFD (2014), and Eurocode 2 did not govern the calculated strengths of the push-through specimens and, therefore, were not evaluated as part of the shear-friction experimental program. The existing limits should be satisfied when performing splice girder designs.

Although the push-through tests provided important insights that can be related to spliced girders, further tests of specimens that include grouted post-tensioning ducts crossing the shear interfaces are recommended and could more closely represent actual splice conditions

6.4 SUMMARY

The shear-friction experimental study conducted to supplement the large-scale splice region tests was described within this chapter. The results of tests performed on eleven push-through specimens were analyzed. Differences in behavior and experimental capacities were shown to result from the various shear interface details that were tested. Furthermore, the additive nature of the strength contributions from the interface reinforcement and the compressive force normal to the shear plane was demonstrated. The shear-friction design provisions in ACI 318-14, AASHTO LRFD (2014), and Eurocode 2 all consider this combined contribution. AASHTO LRFD and Eurocode 2 expressions also include a “cohesion term” that results in notable differences between the performance of these two provisions and ACI 318-14. An evaluation of the push-through specimens using the three design expressions revealed that ACI 318-14 provides the most conservative estimates for the shear capacity. AASHTO LRFD and Eurocode 2 (assuming the shear interfaces of Specimen 1-2 to 2-6 can be classified as “indented”) result in a few unconservative estimates. The performance of the Eurocode 2 provisions is improved if the c- and μ-factors associated with a “rough” surface are used to estimate the strengths of the specimens with shear keys or saw teeth.

In the next chapter, the design recommendations for cast-in-place splice regions are provided. The overall findings of the splice region research program are then summarized in Chapter 8.

Chapter 7. Design Recommendations

7.1 Introduction

Design recommendations for the cast-in-place (CIP) splice regions of spliced I-girder bridges were developed based on the results obtained from the large-scale girder tests. The splice region details of the test girders were carefully selected, and technical input was provided by the TxDOT Project Monitoring Committee (PMC) and a project advisory panel during their development. Two proof tests were then conducted to evaluate the details and to gain insight into the behavior of CIP splice regions. The testing configuration was selected such that the splice regions would experience both high shear and flexural demands in order to study various aspects of the girder behaviors. The results were then used to help develop a comprehensive set of design recommendations. The girders demonstrated satisfactory shear strength and exhibited a failure mechanism similar to that of monolithic post-tensioned girders (Moore, 2014).

Throughout the fabrication and testing of the specimens, observations were made and data was collected that led to the design recommendations presented in the following sections. The recommendations were developed to provide guidelines for satisfactory splice region performance and address issues that can arise during spliced girder construction.

7.2 Shear Strength Calculations at the Splice Region

The test girders exhibited a shear-compression failure of the web concrete with crushing that extended across much of the test span. Comparison of the experimental capacities with calculated shear strengths revealed that both the AASHTO LRFD (2014) general shear provisions and the proposed shear design procedure provide conservative strength estimates for the test girders at the splice regions. Based on these comparisons as well as the observed behaviors of the test girders, the following recommendations were developed for calculating the shear strength at the splice region:

- **Concrete Strength, *f'c*:** The specified compressive strength of concrete, f'_c, used within sectional shear calculations at the splice region should be conservatively defined as the lesser of the specified strengths of the precast concrete and the cast-in-place splice region concrete. As discussed in Section 5.4.4, the governing lower-strength splice region concrete of the test girders was used when evaluating the sectional shear provisions.
- **Effect of Longitudinal Interface Reinforcement:** The splice region behavior observed during the testing program as the specimen capacity was approached provided evidence that mild longitudinal interface reinforcement extending from the precast segments into the splice region should not be considered effective at the ultimate state. It is therefore recommended that the contribution of all interface reinforcement be conservatively neglected in sectional shear strength calculations, as assumed for the evaluation of the shear design provisions in Section 5.4.4.
- **Proposed Modifications to AASHTO LRFD (2014) General Shear Procedure:** The first phase of the spliced girder research program resulted in proposed modifications to the AASHTO LRFD (2014) general shear procedure based on a detailed analysis of the Evaluation Database for Post-Tensioned Girders, or PT Evaluation Database (Moore, 2014). Details of the suggested modifications were provided in Section 2.3.2. In Section 5.4.4, comparisons of the shear strength ratios, V_{test}/V_n, for both the current AASHTO LRFD (2014) general shear provisions and the proposed procedure were presented. The comparisons revealed that, although both design

procedures result in V_{test}/V_n ratios with values greater than 1.0, the proposed modifications provide strength predictions that are slightly more conservative. Considering that the shear strength performance of post-tensioned spliced girders with CIP splice regions can only be evaluated with the two tests of the research program described in this dissertation, it is recommended that the more conservative approach be followed.

To further consider the appropriate shear design procedure for spliced girders in light of the two tests, it is desirable that the shear strength ratios resulting from the evaluation of the spliced girder specimens be consistent with the performance of shear provisions as indicated by the analysis of existing prestressed concrete shear databases. More specifically, the V_{test}/V_n ratios for the splice region tests should ideally be in agreement with the average V_{test}/V_n ratios resulting from the analysis of monolithic prestressed girder test specimens using the AASHTO LRFD (2014) or proposed shear design procedures. Nakamura, Avendaño, and Bayrak (2013) examined the performance of various shear design procedures using tests on prestressed concrete beams within the University of Texas Prestressed Concrete Shear Database (UTPCSDB) that was introduced in Section 2.3.2. Through a filtering process, a more exclusive database was constructed from the UTPCSDB to include only those tests performed on prestressed concrete beams representative of field members and reported to have failed in typical shear failure modes. The resulting set of 171 tests is referred to as the Evaluation database—Level II. Examining the performance of the AASHTO LRFD (2010) general shear procedure using the database of 171 tests gave an average V_{test}/V_n ratio of 1.43 with a coefficient of variation (COV) of 0.18.

Furthermore, as presented in Table 2.1, evaluation of the 41 tests in the PT Evaluation Database with grouted ducts using both the current AASHTO LRFD (2014) general shear procedure and the proposed provisions results in average V_{test}/V_n values of 1.34 and 1.46, respectively. The shear strength ratios for the two spliced girder test specimens considering the AASHTO LRFD general shear procedure were 1.04 and 1.07, while the ratios resulting from the proposed shear design provisions were 1.18 and 1.23 (see Table 5.1). Comparing the two procedures, the V_{test}/V_n values from the proposed provisions are in better agreement with the average V_{test}/V_n values resulting from the database evaluations. Moreover, the shear-compression failure mechanism exhibited by the spliced girder specimens is consistent with the mechanical model on which the proposed modifications are based (Moore, 2014).

For the reasons outlined above, the use of the proposed modifications to the AASHTO LRFD (2014) general shear procedure is recommended for calculating the nominal shear resistance, V_n, of spliced girders.

The interface shear strength at the splice region interface is also a limit state that should be considered when designing spliced girders. Designers should ensure that splice regions have adequate interface shear strength according to the applicable provisions. The spliced girder specimens of the testing program were designed to evaluate sectional shear provisions, and interface shear failures did not occur. Significant insights into shear-friction behavior, however, were gained through the push-through tests described in Chapter 6. The experimental program included comparisons of the shear-friction behavior resulting from various shear interface details (see Section 6.3.4).

7.3 SPLICE REGION DETAILS

Recommendations for splice region details are presented in this section. Drawings of a splice region with the proposed details are provided in Appendix D.

7.3.1 Longitudinal Interface Reinforcement

The primary variable between the two test girders was the longitudinal interface reinforcement. As discussed in Sections 5.4.2 and 5.5, the additional interface reinforcement included within the bottom flange of Test Girder 2 (approximately 3 times the interface reinforcement within the bottom flange of Test Girder 1) resulted in improved behavior after the initiation of flexural cracking. Based on the performance of the two specimens, the additional area of reinforcement provided in the second test girder is recommended (area of reinforcement in the bottom flange was equal to 0.78 percent of the flange area). The interface reinforcement details of Test Girder 2 are again provided in Figure 7.1 for easy reference. Please refer to Appendix D for detailed splice region drawings.

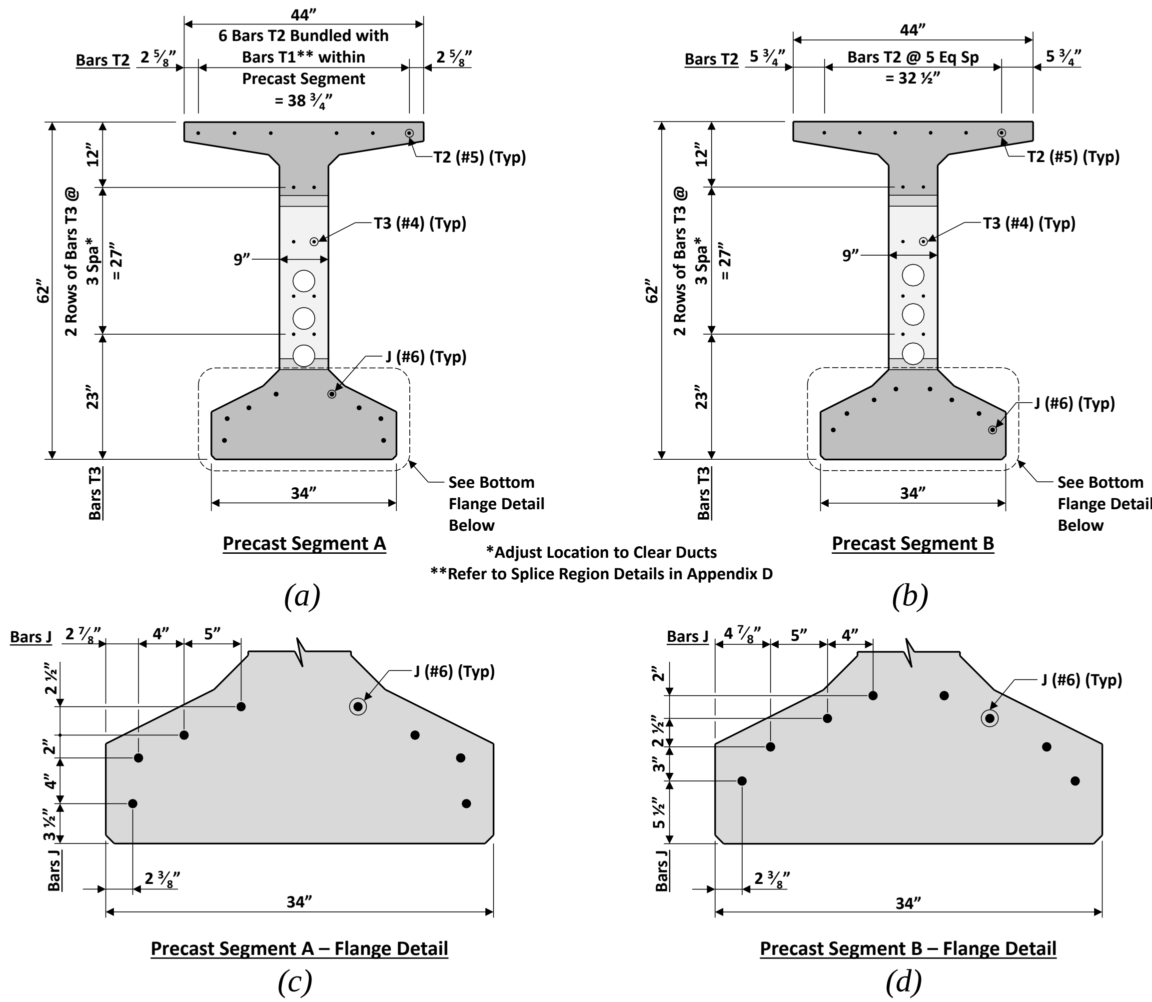


Figure 7.1: Recommended longitudinal interface reinforcement– (a) end of Precast Segment A; (b) end of Precast Segment B; (c) flange detail of Precast Segment A; (d) flange detail of Precast Segment B

As described in Section 5.4.2, flexural cracking at the splice region can be prevented under service-level design loads by ensuring that longitudinal tensile stress due to service loads does not overcome the compressive stress caused by the post-tensioning force (long-term effects should be considered in design). Should flexural cracks form, the recommended interface reinforcement was selected with the intent to better control cracking when compared to a lesser amount of reinforcement such as that of Test Girder 1.

To aid in the development of the longitudinal interface reinforcement within the splice region after the formation of cracks, various alternatives to the details presented in Figure 7.1 could be explored when designing spliced girders. For example, hairpin bar details, as presented in Section 3.3.5, could be specified. Lap splices designed in accordance with Article 5.11.5 of AASHTO LRFD (2014) can be used if continuity is desired. The use of confinement reinforcement within the bottom flange of the splice region as illustrated in Figure 7.2 could also be considered in order to improve the development of the mild steel and better control the horizontal cracking behavior of the bottom flange observed during the testing program (see Section 5.5). With any detail that is specified, however, the designer should remain mindful of potential rebar congestion within the splice region. The design should allow for proper concrete consolidation, and special consideration should be given to concrete placement in the bottom flange.

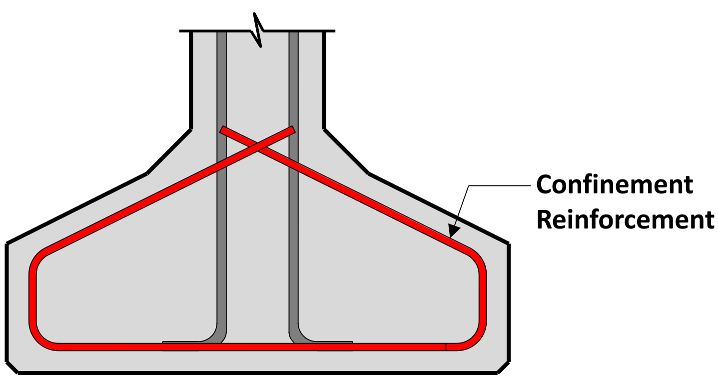


Figure 7.2: Potential confinement reinforcement within the splice region

Please recall that the pretensioned strands extending from the ends of the precast girder segments were cut within approximately 3 in. of the girder faces to avoid additional congestion within the splice regions. A similar detail is recommended in the drawings provided in Appendix D.

7.3.2 Splice Region Geometry

The splice region geometry for the girder specimens was discussed in Sections 4.9.1 and 4.9.2. The length of the splice region measured along the longitudinal axis of the girders was chosen to be 24 in. From the first-hand experience of constructing the splice regions of the test specimens, a length of 24 in. is recommended from a constructability standpoint. The recommended length allowed the ducts to be coupled while accommodating any minor duct misalignment issues. Moreover, the 24-in. length was needed to ensure adequate space for reinforcing bar placement. The splice region fabrication would have been more difficult if a shorter length had been chosen. Please note that the splice region length also determines the space available for the splicing of longitudinal reinforcing bars (Article 5.14.1.3.2b of AASHTO LRFD (2014)).

The cross-section at the splice regions of the test specimens was selected to match the shape of the adjacent precast girder segments. Although this choice resulted in a restricted space for concrete placement, the splice region was successfully cast by ensuring adequate vibration of the concrete (see Section 4.13.3). Furthermore, the girders exhibited satisfactory shear strength when compared to calculated values and displayed a shear-compression failure mechanism consistent with monolithic post-tensioned girders (Moore, 2014). Based on these results, the recommended splice region details maintain a constant cross-section for the precast segments and the splice region. This recommended geometry may also be desirable for aesthetic reasons to provide a constant shape along the length of a fascia girder.

7.3.3 Other Splice Region Details

A shear key was included at the interface of the splice regions of the test specimens (refer to Section 4.9.3). The selected detail exhibited satisfactory strength and serviceability performance during both proof tests. The shear key detail is therefore included in the recommended splice region details in Appendix D.

The mild shear and transverse reinforcement placed in the splice regions of the test girders was essentially a continuation of reinforcement provided in the precast segments (refer to Section 4.9.6). Based on the strengths of the test specimens, it is recommended that the shear reinforcement within the splice region be the larger of that provided in the adjacent precast girders, as is currently required in the AASHTO LRFD (2014) provisions (see Article 5.14.1.3.2b). The designer must ensure that clear cover requirements for the shear reinforcement are satisfied considering the combination of the web width and duct diameter that is specified. The size of the duct couplers should also be considered when checking clear cover.

The duct coupling detail that was selected for the splice region consisted of two couplers with a short duct segment in the middle (refer to Section 4.9.7). From a constructability standpoint, the coupling detail accommodated any minor duct misalignment issues. As mentioned in Section 5.3, however, the formation of shear cracks during the testing program may have been influenced by the locations of the duct couplers. It is important to note that the various types of couplers available in the market may result in different cracking behaviors. The duct coupling detail (i.e., the use of one or two couplers for each duct) is therefore left to the discretion of the engineer.

7.4 SUMMARY

Design recommendations based on the results of the large-scale experimental program were presented in this chapter. The recommended splice region details were selected based on the performance observed during the proof tests while minimizing potential constructability issues. Guidelines for spliced girder shear strength calculations were also presented. The results of the experimental program reinforce the need to modify the AASHTO LRFD (2014) general shear procedure as outlined in Chapter 2. The proposed modifications are based on a mechanical model consistent with the behavior of post-tensioned bridge girders, including the spliced girder specimens.

The two proof tests described in this dissertation provided significant insights into the strength and serviceability behavior of spliced girders. The test results also led to recommendations that can be directly applied to the design and detailing of field structures. Nevertheless, an evaluation of all the provisions relating to the design and detailing of spliced girders within the AASHTO LRFD (2014) specifications was beyond the scope of this research, and additional testing and evaluation of spliced girder behavior could result in further refinements to the splice region details.

Chapter 8. Summary and Conclusions

8.1 SUMMARY

The second phase of the spliced girder research program was developed to build upon the findings of the first phase of the project, which focused on the shear performance of monolithic post-tensioned girders. In order to better understand the behavior of the cast-in-place (CIP) splice regions of spliced I-girder bridges, two large-scale tests were performed as part of the second phase of the research and provided invaluable information regarding splice region behavior that is otherwise unavailable in the literature. The primary objectives of the CIP splice region research were:

(i) Study the strength and serviceability behavior of the CIP splice regions of spliced I-girders.

(ii) Identify design and detailing practices that have been successfully implemented in CIP splice regions located within the span lengths of existing spliced I-girder bridges.

(iii) Develop design recommendations based on the structural performance of spliced girder test specimens that include details typical of current practice within the CIP splice regions.

Considering the wide variety of splice region details that have been used in the field, the identification of best practices that have been successfully implemented for splice regions of existing bridges was needed. During this process, awareness of potential constructability issues relating to each detail was essential. An industry survey was therefore conducted to aid in the selection of splice details that would later be included in spliced girder specimens and proof tested. In addition to the survey, the TxDOT Project Monitoring Committee (PMC) and a project advisory panel offered invaluable input from first-hand experience with spliced girder technology. With these available resources, splice region details were developed and implemented in two I-girder test specimens.

Each test girder consisted of two precast pretensioned segments that were joined with a 2-ft long CIP splice region. The girders were made continuous by three post-tensioning tendons that extended the full length of the specimens. Due to the interest in studying the shear strength behavior of spliced girders, the specimens were designed to fail in shear. As the ultimate load was approached during the tests, the girders also experienced high flexural demands, providing the opportunity to study various aspects of the splice region behavior and enhancing the value of the proof tests. The amount of mild longitudinal interface reinforcement extending from the precast segments into the splice region was varied between the two specimens to identify the effect of the bars on the behavior of the girders after the initiation of flexural cracking at the splice regions.

The two proof tests were conducted successfully with the girders exhibiting shear-compression failure mechanisms similar to that of the monolithic post-tensioned specimens of the first phase of the spliced girder research program. From the test results, design recommendations for splice region details and shear strength calculations were developed. The recommendations were outlined in Chapter 7.

A shear-friction experimental program was conducted to supplement the spliced girder research and provide a deeper insight into interface shear transfer at CIP splice regions. Detailed observations from the push-through tests along with an evaluation of existing shear-friction design provisions were presented in Chapter 6.

8.2 Observations and Conclusions

The overall findings of the CIP splice region research program are outlined in this section. Key observations collected during the proof tests and shear-friction study are included along with the conclusions developed from the analysis of the research results.

8.2.1 Behavior of the Splice Regions of Spliced I-Girder Bridges

- **Service-Level Behavior:** During the load tests, the development of localized shear cracks were observed in the vicinity of the post-tensioning ducts at service-level shear forces. The cracking continued to distribute within the vicinity of the ducts upon further loading. The formation of cracks that extended over much of the web depth was visually detected at shear forces of 73 percent and 69 percent of the maximum shear force, V_{test}, for the first and second test girders, respectively, consistent with behavior observed during the monolithic girder tests (Moore, 2014).

  Considering the flexural behaviors of the girders, it was demonstrated that flexural cracking can be prevented under service-level design loads (refer to Section 5.4.2).
- **Shear Behavior at Failure:** Both girder specimens exhibited a shear-compression failure mechanism characterized by crushing that occurred primarily in the vicinity of the top post-tensioning duct. This behavior was consistent with the observed failures of monolithic post-tensioned I-girders (Moore, 2014). The web crushing of the spliced girder specimens extended across much of the test span and was not localized within the splice regions.
- **Recommended Shear Strength Calculations:** Comparison of the experimental capacities with calculated shear strengths revealed that both the AASHTO LRFD (2014) general shear procedure and the proposed shear design provisions (see Appendix E) provide conservative strength estimates for the test specimens at the splice regions. Application of the proposed sectional shear design procedure is recommended for spliced girder strength calculations. When calculating the shear resistance, the lesser of the specified

strengths of the precast concrete and the cast-in-place concrete should be assumed as the value of f'_c at the splice region interface. Furthermore, any contribution of mild longitudinal interface reinforcement should be neglected.

8.2.2 Splice Region Details

- **Industry Survey Results**: The industry survey results provided insights into the design and construction of spliced girder bridges from the viewpoints of state Departments of Transportation (DOTs) with experience in spliced girder technology. The survey was designed to identify splice region details that are typically specified in each state. Although the details vary significantly among the state DOTs, successful practices could be identified from the survey responses and supplementary material provided by the participants. In addition to the industry survey, input from the TxDOT Project Monitoring Committee and the project advisory panel were invaluable when developing the splice region details to be tested.
- **Large-Scale Splice Region Proof Tests**: The results of the load tests on the spliced girder specimens were analyzed to evaluate the performance of the selected splice region details. Both test girders experienced shear forces at the critical section (i.e., splice region interface) that were greater than the calculated strengths based on sectional shear provisions. The chosen splice region details also allowed both specimens to exhibit failure behaviors similar to that of monolithic girders (i.e., concrete crushing was not localized at the splice region).
- **Recommended Splice Region Details**: Splice region details were proposed based on the structural performance of the test girders and other relevant observations that were gathered during the research program. The

recommended mild interface reinforcement was based on the cracking behavior of the two test girders. The second test girder with a larger area of interface reinforcement compared to Test Girder 1 exhibited better cracking behavior at high loads. Lessons learned during the construction of the splice regions also contributed to the recommendations. Drawings of the recommended splice region details are provided in Appendix D.

8.2.3 Interface Shear Transfer at Splice Regions

- **Effect of Shear Interface Details**: The push-through test specimens of the shear-friction study were designed with various interface details to determine their effect on shear performance. The tests indicated that shear keys or saw teeth result in interlocking action that contributes to increased interface shear strength when compared to the strength of a specimen with a smooth interface.
- **Influence of Interface Reinforcement and Post-Tensioning Force**: The shear-friction test results confirmed the additive contribution of interface reinforcement and post-tensioning force to interface shear strength as expressed by the sum $A_{vf}f_y + P_c$.
- **Performance of Shear-Friction Design Provisions**: The shear-friction design expression in ACI 318-14 provided conservative strength estimates for all eleven push-through specimens. With the absence of a cohesion term in the expression, however, some of the strength predictions could be considered overly-conservative. The provisions resulted in an average shear strength ratio, V_{test}/V_n, of 1.67, with a maximum ratio of 2.11. The AASHTO LRFD (2014) shear-friction expression provided relatively accurate strength predictions, but the strengths of three test specimens were unconservatively

estimated. The values of V_{test}/V_n, however, were 0.92 or greater in all cases. The interface shear provisions in Eurocode 2, applying the factors for a "rough" surface to calculate the strengths of the interfaces with shear keys or saw teeth, arguably resulted in the most favorable performance, with no unconservative estimates and an average V_{test}/V_n of 1.42.

8.3 Concluding Remarks

The spliced girder research program resulted in a better understanding of the shear behavior of spliced post-tensioned girders. During the first phase of the project (detailed in Moore (2014)), eleven shear tests were performed on large-scale I-girder specimens. Ten of these tests were added to the Evaluation Database for Post-Tensioned Girders, which now contains a total of 44 tests. The specimens from the spliced girder research therefore comprise 23 percent of the database. A comprehensive analysis of the database resulted in proposed modifications to the AASHTO LRFD (2014) general shear procedure to better account for the presence of post-tensioning ducts in the webs of bridge girders. The second phase of the spliced girder research program (described in this dissertation) included the only known tests in which a shear failure mechanism was developed in spliced girders containing post-tensioned in-span cast-in-place splice regions. The two spliced girder tests lead to a better understanding of CIP splice region behavior. The spliced girders failed at shear strengths exceeding calculated values. Furthermore, splice region details were recommended based on the results of the research program. The spliced girder research findings are presented with the hope that their implementation will provide the tools necessary for the precast concrete girder industry to expand the use of precast construction.

Appendix A. Industry Survey Responses

INTRODUCTION

The responses to the industry survey conducted for the spliced girder research program are provided in this appendix. A total of 25 responses were received. The survey participants who indicated that their state/district did not have past experience with the design and/or construction of spliced girder bridges were not required to proceed with the survey after the second page. In these cases, only the first two pages of the survey responses are included.

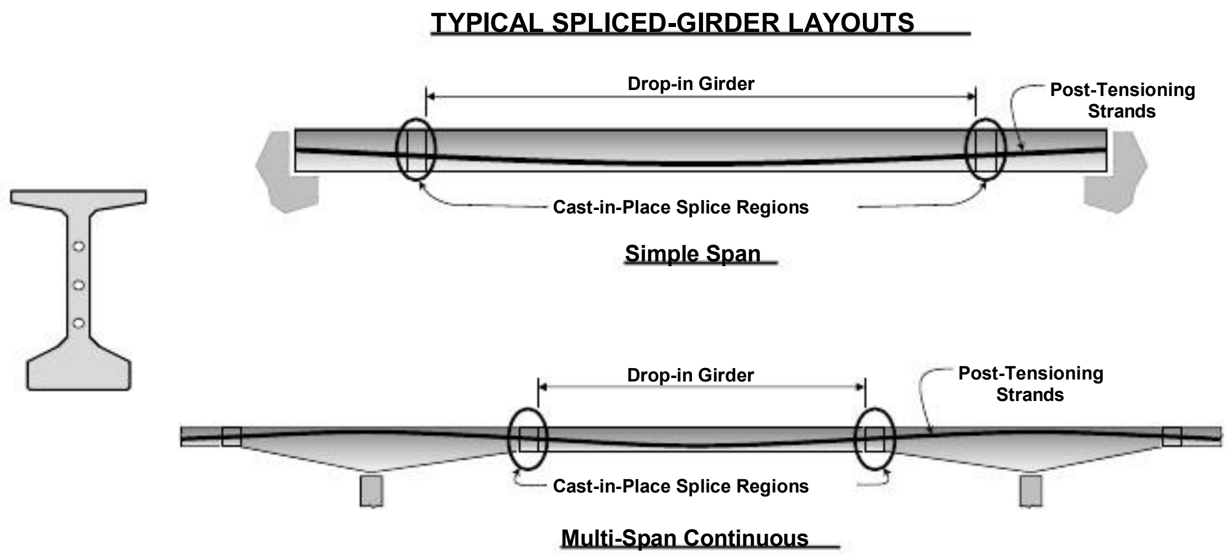
Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:	The University of Texas Research Team:
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: __________

Title: __________

State/District: Arizona

Organization/Unit: Arizona Department of Transportation

Address: __________

Phone: __________

Fax: __________

Email: __________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

☒ Yes ☐ No

If *No,* has your state/district considered the use of spliced girder technology?

☐ Yes ☐ No

If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

____

If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

☐ None ☒ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

☐ None ☒ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ■ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

■ Yes ☐ No

If *Yes,* please list the consultant(s).

____

____

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 100 %
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

____

____

____

____

____

____

____

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ■ No

If *Yes,* please briefly describe the problem(s).

____

____

____

____

____

____

____

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

[x] Yes [ ] No

If *Yes,* what design provisions are used to calculate the strength reduction?

[x] AASHTO LRFD Specifications [ ] AASHTO Segmental Bridge Specifications

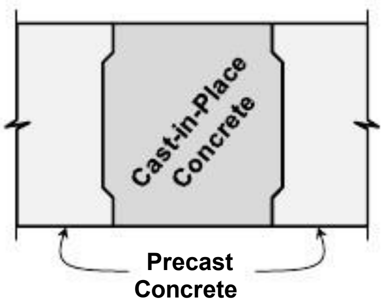
[ ] Other; please specify: ______________________________

9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

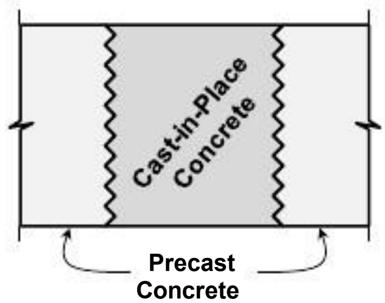
[ ] Yes [x] No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

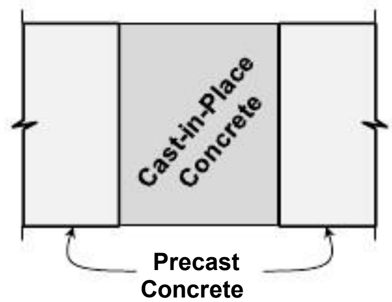
[ ] Shear key: 80 %



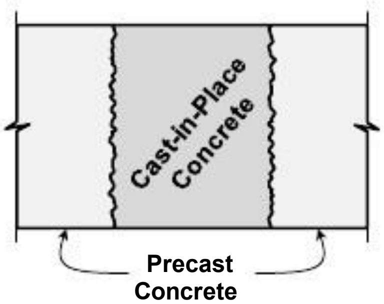
[x] Saw teeth: 20 %



[ ] Plain: ________%



[ ] Sandblasting or intentional roughening: ________%

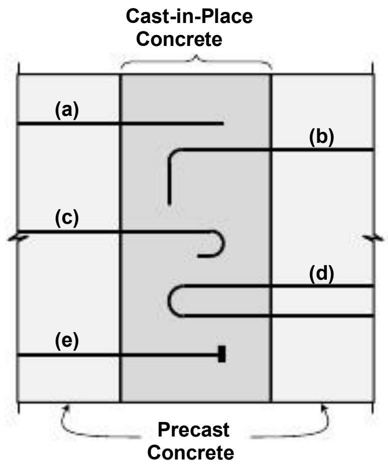


Please explain the factors that affect the type of interface that is chosen.

designer determines

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

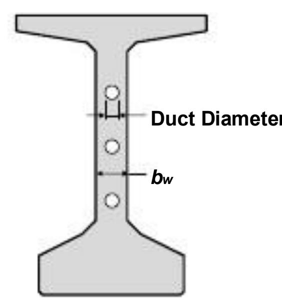
- ☐ (a) Straight bars
- ☐ (b) 90-degree hooks
- ☐ (c) 180-degree hooks
- ■ (d) Hairpins
- ☐ (e) Headed bars
- ☐ Other; please describe: ______________________



Please elaborate on the detailing of the interface reinforcement if necessary.

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
8	4"	100 %
		%
		%
		%
		%

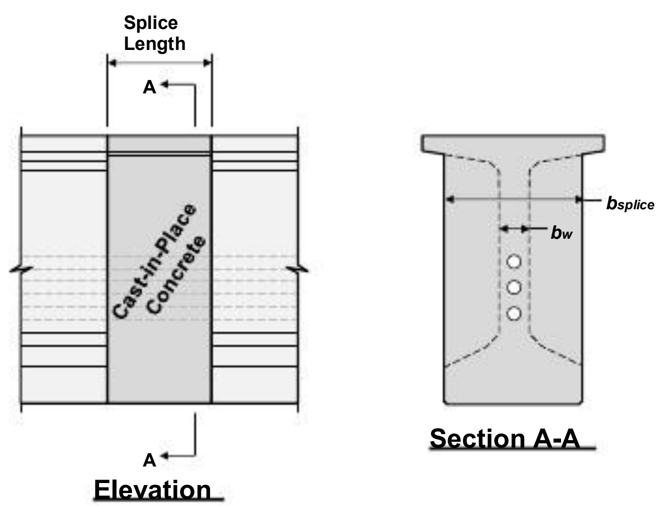


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

■ At the splice ☐ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., b_{splice} - b_w), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the (b_{splice} - b_w) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	24"		24"
b_{splice} - b_w	16"		16"



Please explain the factors that affect the **length** of the splice region?

the need to lap reinforcement

Please explain the factors that affect the **width** of the splice region?

Typical use the width of the bottom flange of the precast girder or width of the web of precast box girders

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

Two of the sliced AASHTO girder bridge (both two spans) have slight angle point at splice locations because roadway was on a slight curve.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here:

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

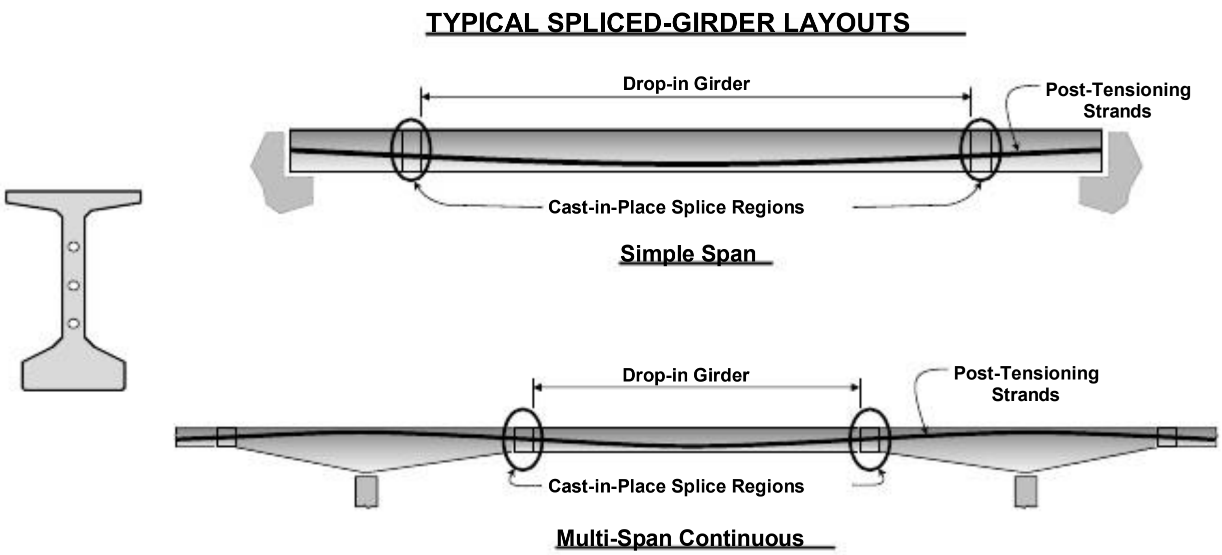
Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:
Greg Turco, PE

Address:
Bridge Division

The University of Texas Research Team:
Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu
Dr. James Jirsa: jirsa@uts.cc.utexas.edu
Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu
Chris Williams: chrisw05@utexas.edu
Andy Moore: ammoore@utexas.edu

Address:
Ferguson Structural Engineering Laboratory
The University of Texas at Austin
10100 Burnet Rd., Building 177
Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ____________________

Title: ____________________

State/District: Arkansas

Organization/Unit: Arkansas State Highway and Transportation Department

Address: ____________________

____

Phone: ____________________

Fax: ____________________

Email: ____________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☐ Yes ☒ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☒ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   Arkansas does not have a concrete beam fabricator so a small percentage of the bridges utilize concrete beams.

   Spliced girder technology has not been used due to our unfamiliarity with the process.

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

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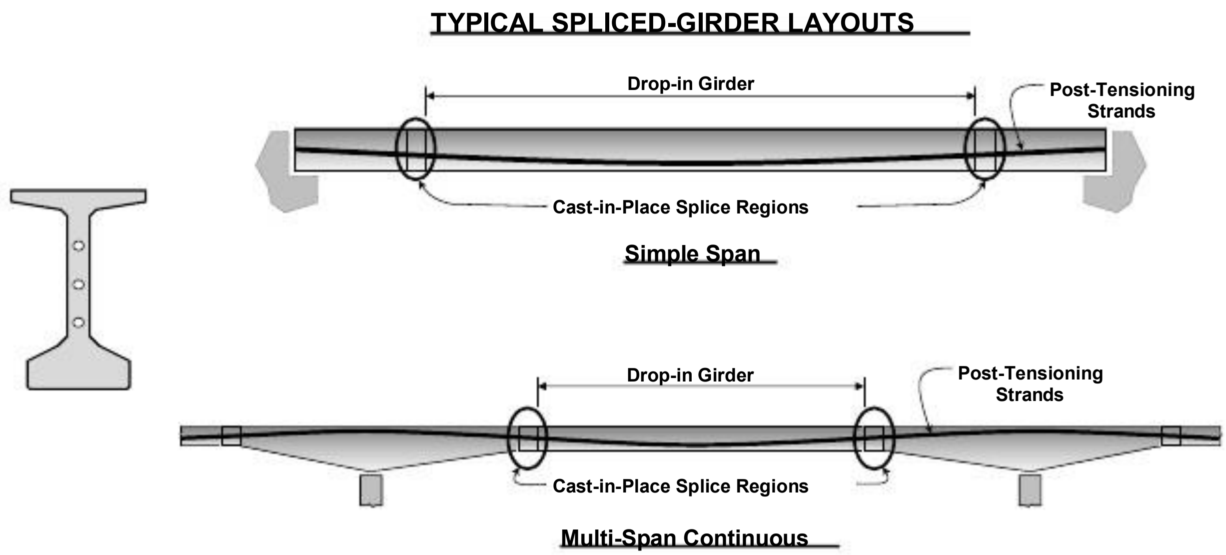

Texas Department of Transportation Contact:	The University of Texas Research Team:
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: California

Organization/Unit: California Department Of Transportation

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☒ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

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   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☒ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☒ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ☒ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

☒ Yes ☐ No

If *Yes,* please list the consultant(s).

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 100 %
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

CALTRANS DOES NOT ALLOW PLASTIC DUCTS IN POST-TENSIONING SYSTEMS.

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ☒ No

If *Yes,* please briefly describe the problem(s).

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

☒ Yes ☐ No

If *Yes,* what design provisions are used to calculate the strength reduction?

☒ AASHTO LRFD Specifications ☐ AASHTO Segmental Bridge Specifications

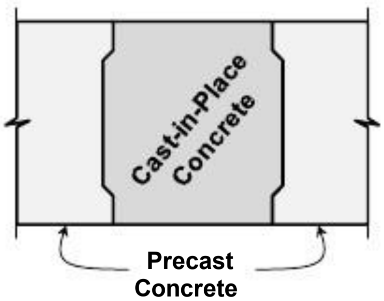
☐ Other; please specify: CALIFORNIA AMENDMENTS

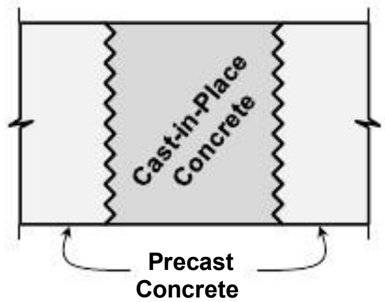
9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

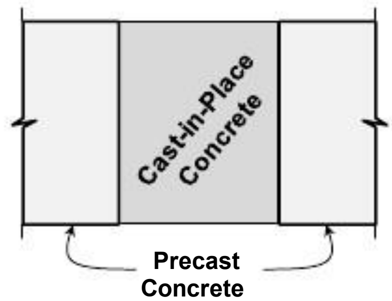
☐ Yes ☒ No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

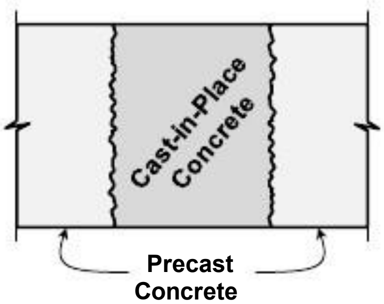
☐ Shear key: 30 % ☐ Saw teeth: ____% ☐ Plain: 50 %







☐ Sandblasting or intentional roughening: 20 %

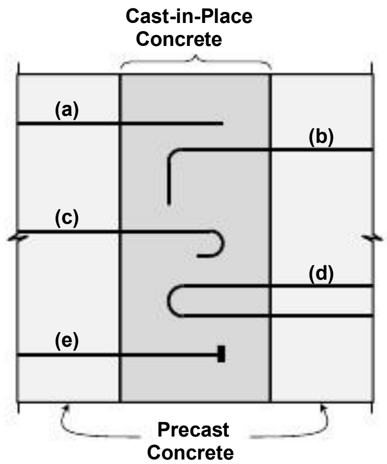


Please explain the factors that affect the type of interface that is chosen.

CALTRANS HAS MOVED FROM PLAIN INTERFACE DETAIL TO SHEAR KEY FACE AND ROUGH INTERFACE DETAILS.

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

- ☐ (a) Straight bars
- ☒ (b) 90-degree hooks
- ☐ (c) 180-degree hooks
- ☐ (d) Hairpins
- ☐ (e) Headed bars
- ☐ Other; please describe: ______________________



Please elaborate on the detailing of the interface reinforcement if necessary.

____

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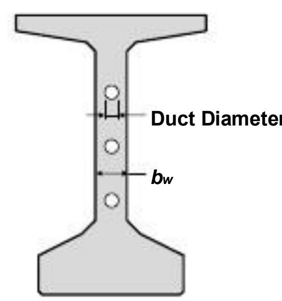
12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
8"	4"	30 %
8"	3.5"	50 %
8"	3"	20 %
		%
		%

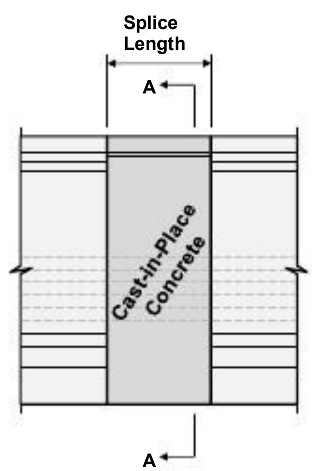


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

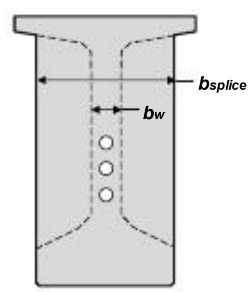
- ☐ At the splice
- ☒ Away from the splice
- ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	24"	48"	24"
$b_{splice} - b_w$	21.5"	38.5"	bsplice is the SAME WIDTH AS THE BULB



Elevation



Section A-A

Please explain the factors that affect the **length** of the splice region?

1. PT DUCT SPLICE LENGTH
2. DEVELOPMENT LENGTH OF THE EXTENDED STRANDS AND REBARS
3. SPACE FOR SHEAR REINFORCEMENT
4. ROOM FOR WORKING SPACE

Please explain the factors that affect the **width** of the splice region?

1. DOABLE SECTION OF PRECAST GIRDER FORM
2. ENOUGH SPACE FOR REBAR PLACEMENT
3. ENOUGH SPACE FOR CONCRETE POUR AND VIBRATION

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☒ Yes ☐ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

ISSUE: CONCRETE AIR POCKETS AND VOIDS AT SPLICE REGIONS
PROBLEM CAUSED: DIDN'T (AND HARD TO) VIBRATE CONCRETE ENOUGH FOR DEEP GIRDER BRIDGES
LESSONS LEARNED: IMPROVE VIBRATION METHOD AND REMOVE FORMWORK TO INSPECT THE SPLICE REGIONS BEFORE POST-TENSIONING

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

ONE OF THE NOTES FOR SPLICED GIRDER DESIGN REGARDING TO SPLICE REGIONS IS: Wet closure joints between
girder segments are usually used instead of match-cast joints. The width of a closure joint shall not be less than 24 inches and shall allow for the splicing of post tensioning ducts and rebar. Web reinforcement (Av/s) within the joint should be the larger of that provided in the adjacent girders. The face of the precast segments at closure joints must be intentionally roughened or cast with shear keys in place.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here: WILL ATTACH DESIGN GUIDELINES AND UPLOAD SOME DESIGN PLANS

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

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Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

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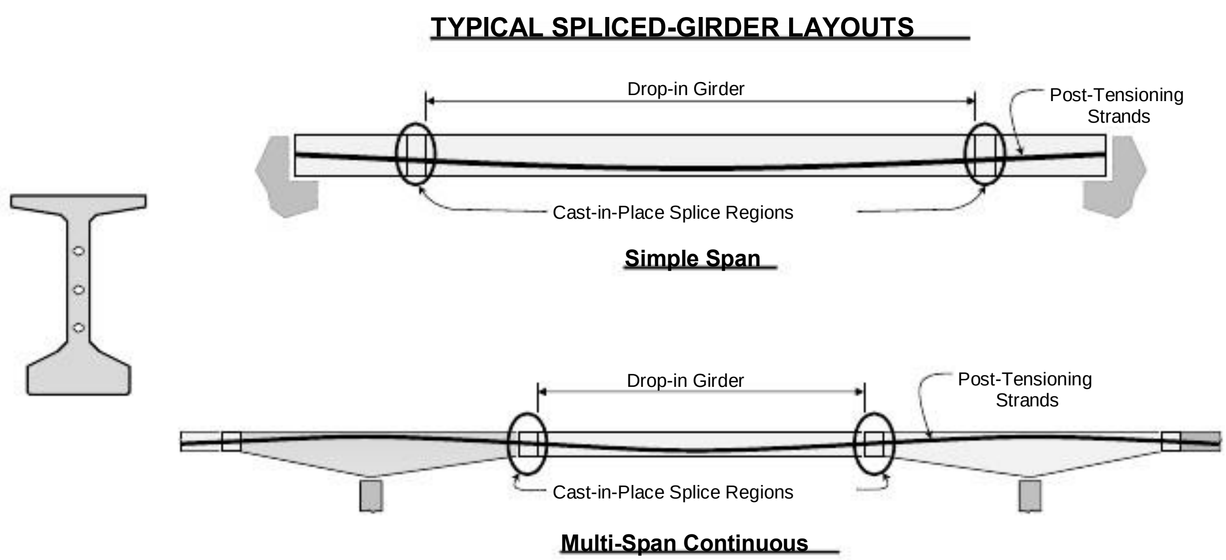

Texas Department of Transportation Contact:	**The University of Texas Research Team:**
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ________________

Title: ________________

State/District: Washington, DC

Organization/Unit: DC Department of Transportation

Address: ________________

Phone: ________________

Fax: ________________

Email: ________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☐ Yes ☒ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☒ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

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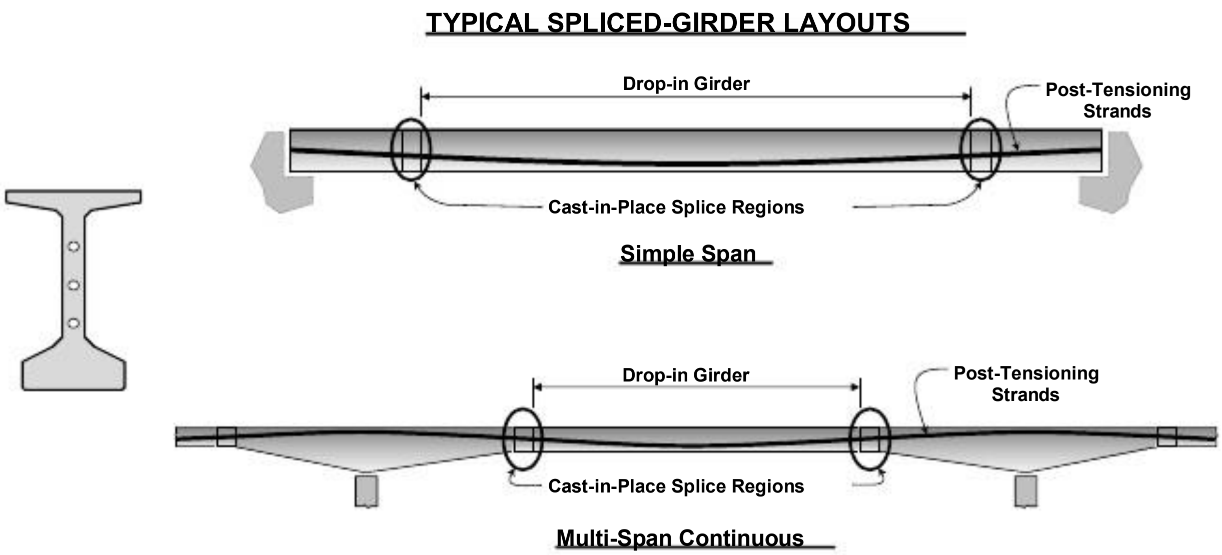

Texas Department of Transportation Contact:	The University of Texas Research Team:
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Florida/Central Office (Tallahassee)

Organization/Unit: Structures Design Office

Address: ______________________

____

Phone ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☒ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

   ____

   ____

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   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☒ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☒ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ■ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

■ Yes ☐ No

If *Yes,* please list the consultant(s).

There are currently 52 Consultant Firms qualified to design spliced I-girder bridges in the State of Florida. Spliced I-girders are covered under Work Group 4.2.1, Major Bridge Design - Concrete as defined in Rule 14-75. See the URL link below for qualifying firms and Rule 14-75.
http://www2.dot.state.fl.us/procurement/ProfessionalServices/lppc/prequal_listing.asp
http://www.dot.state.fl.us/procurement/Project%20Costing/pdf/Rule%2014-75.pdf

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 60 %
- Plastic (HDPE): 40 %

If one of the materials is preferred over the other, please explain why.

Corrugated steel duct was used on older structures. New FDOT policy is to use corrugated polypropylene (not HDPE) for internal tendons.

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ■ No

If *Yes,* please briefly describe the problem(s).

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

☐ Yes ☒ No

If *Yes,* what design provisions are used to calculate the strength reduction?

☐ AASHTO LRFD Specifications ☐ AASHTO Segmental Bridge Specifications

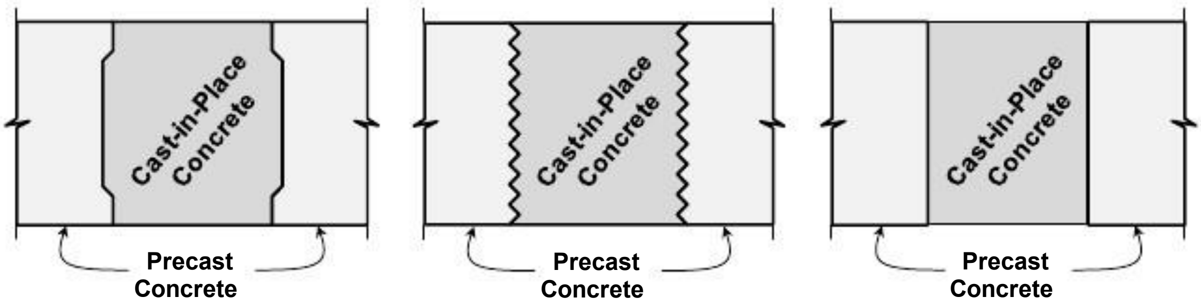
☐ Other; please specify: ______________________________

9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

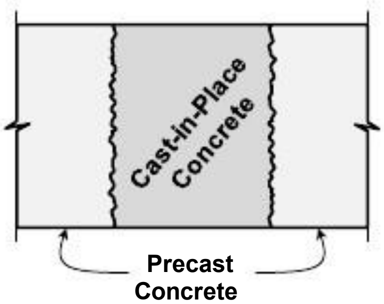
☐ Yes ☒ No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

☒ Shear key: 100 % ☐ Saw teeth: ________% ☐ Plain: ________%



☐ Sandblasting or intentional roughening: ________%



Please explain the factors that affect the type of interface that is chosen.

____
____
____
____
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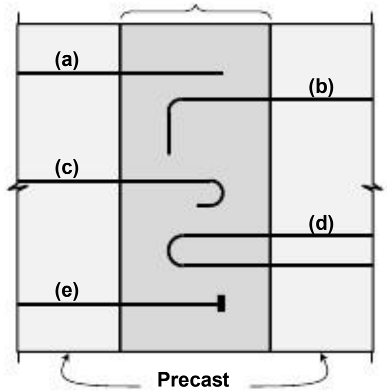
____
____

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

- [ ] (a) Straight bars
- [ ] (b) 90-degree hooks
- [x] (c) 180-degree hooks
- [x] (d) Hairpins
- [ ] (e) Headed bars
- [ ] Other; please describe: ______________________________



Please elaborate on the detailing of the interface reinforcement if necessary.

____

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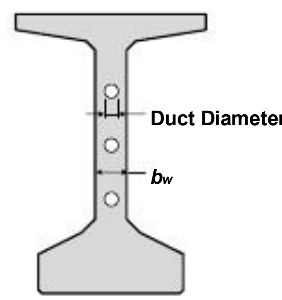
____

____

____

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
8" (+/-)	4" (Steel)	%
7"	2 3/8" x 5"(PE) (Oval)	%
8 1/2"	4" (PP)	%
9"	4" (PP)	%
		%

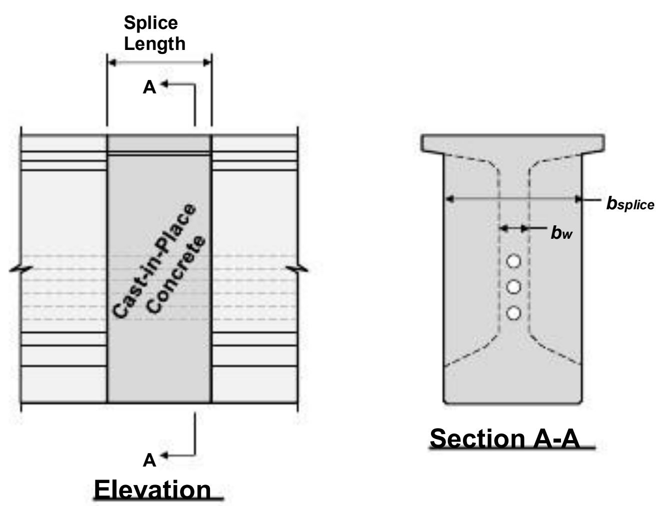


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

■ At the splice ☐ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	18" (+/-)	20" (+/-)	N/A
$b_{splice} - b_w$			



Please explain the factors that affect the **length** of the splice region?

Length to make duct connections

Reinforcing details

Please explain the factors that affect the **width** of the splice region?

Typically, diaphragms are used in the splice locations. Width of splice region is the same as the length of the diaphragm.

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

We have had only one instance when the drop in segment "walked off" the supports. This has been addressed in the SDG Detailing Manual Chapter 23.
(http://www.dot.state.fl.us/Structures/StructuresManual/CurrentRelease/Vol2_SDM.pdf)

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

-New Chapter in the Structures Manual for Spliced Girder Construction (see Chapter 23 in the Detailing Manual)
-Developed policy for sizing webs for spliced girders. See Table 4.5.6 of the Structures Design Guidelines in the Structures Manual. (http://www.dot.state.fl.us/Structures/StructuresManual/CurrentRelease/Vol1_SDG.pdf)
-Developed maximum duct dimensions to be used for detailing. See Table 4.5.12-1 of the Structures Design Guidelines in the Structures Manual.
-Curved U-Girder bridges are currently under design for expressway authorities.
A new design bulletin which outlines our policies for spliced curved U-beams will be issued by the end of May, 2013.
For a list of spliced I-girder bridges constructed in Florida prior to 2004, see NCHRP, Report 517, Appendix C2.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here: ______

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

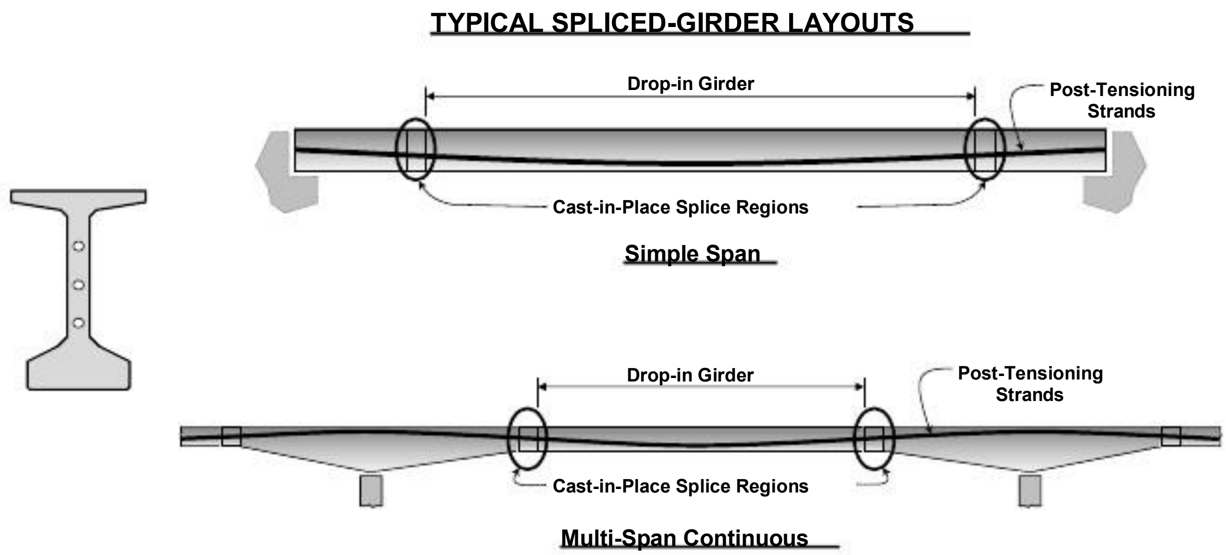
Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:	**The University of Texas Research Team:**
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Georgia

Organization/Unit: Georgia Department of Transportation

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☒ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

   ____

   ____

   ____

   ____

   ____

   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☒ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☒ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ■ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

■ Yes ☐ No

If *Yes,* please list the consultant(s).

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 50 %
- Plastic (HDPE): 50 %

If one of the materials is preferred over the other, please explain why.

HDPE is less prone to corrosion and it is felt that the HDPE ducts can be sealed better.

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

■ Yes ☐ No

If *Yes,* please briefly describe the problem(s).

We had leakage in the ducts for our metal duct spliced girder bridge.

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

☐ Yes ☒ No

If *Yes,* what design provisions are used to calculate the strength reduction?

☐ AASHTO LRFD Specifications ☐ AASHTO Segmental Bridge Specifications

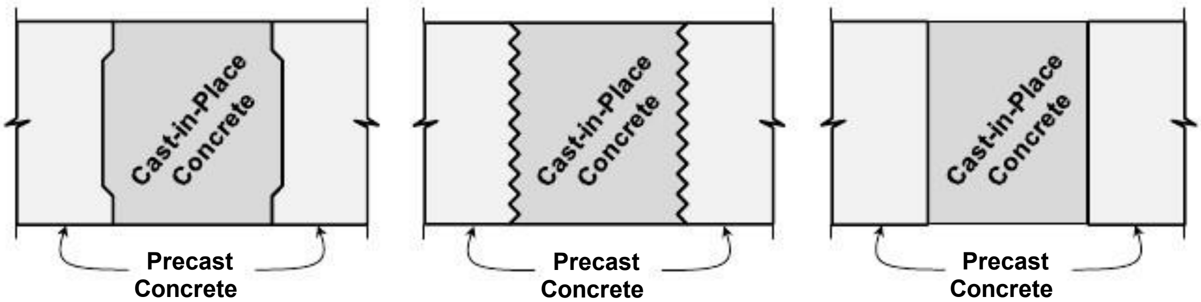
☐ Other; please specify: ______________________________

9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

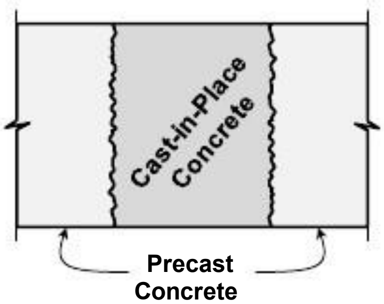
☐ Yes ☒ No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

☒ Shear key: 100 % ☐ Saw teeth: ________% ☐ Plain: ________%



☐ Sandblasting or intentional roughening: ________%

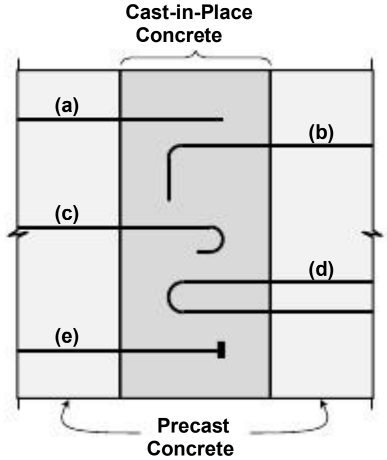


Please explain the factors that affect the type of interface that is chosen.

Shear keys are usually required in the design specs. The shape of the shear keys is usually the Designer's decision

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

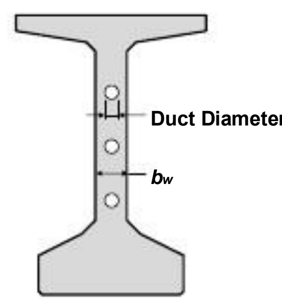
- ☐ (a) Straight bars
- ■ (b) 90-degree hooks
- ■ (c) 180-degree hooks
- ■ (d) Hairpins
- ☐ (e) Headed bars
- ■ Other; please describe: No reinforcement, uses a stepped joint



Please elaborate on the detailing of the interface reinforcement if necessary.

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
9"	3.82"	50 %
12"	2" duct pairs	50 %
		%
		%
		%

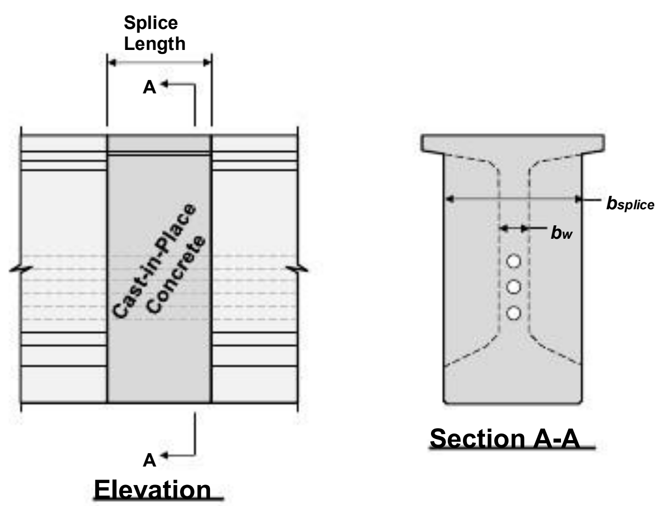


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

☐ At the splice ☒ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	Br 1 - 4", Br 2 - 2'-0"	Br 1 - 6", Br 2 - 2'-6"	Br 1 - 4", Br 2 - 2'-0"
$b_{splice} - b_w$	Br 1 - 1'-2", Br 2 - 0"	Br 1 - 1'-2", Br 2 - 0"	Br 1 - 1'-2", Br 2 - 0"



Please explain the factors that affect the **length** of the splice region?

The shape of the splice- See our two examples

Please explain the factors that affect the **width** of the splice region?

The width of the I-beam that is being spliced and weather there are any built-up areas in the vicinity of the splice.

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☒ Yes ☐ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

Leakage of the metal ducts.

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

We have two spliced I-girder bridges. The Talmadge Memorial Bridge approaches built in the early 1990's and the Skidaway Narrows Bridge that is being constructed now. Both are in Savannah, Georgia. This a viable structural design method but it usually difficult to construct in the field. Plans are being sent under another e-mail.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here: ______

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

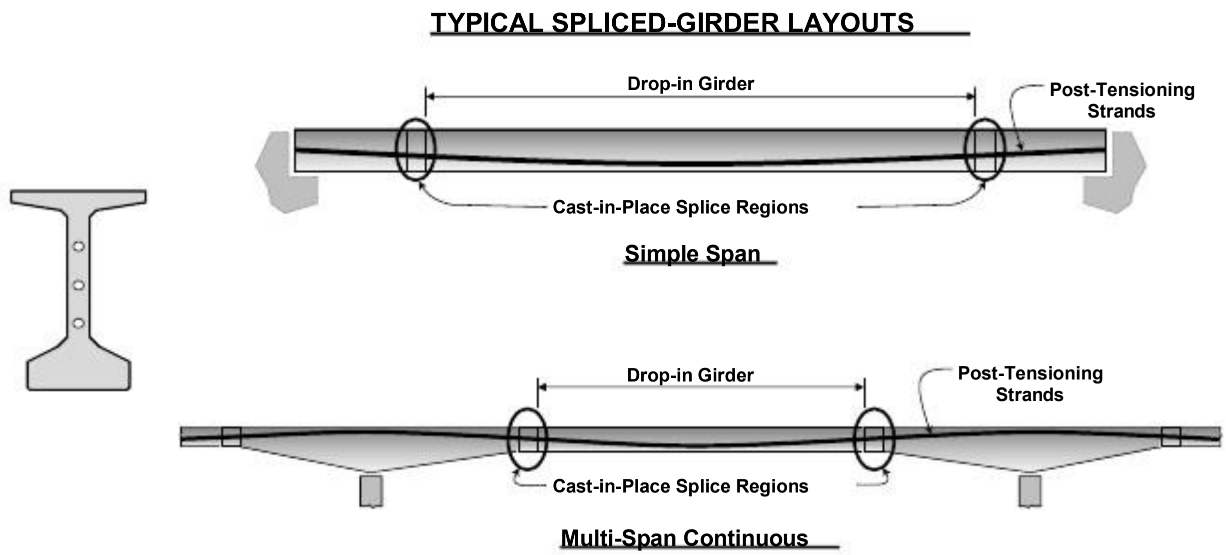
Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:	The University of Texas Research Team:
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Hawaii

Organization/Unit: Hawaii DOT, Bridge Design Section

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☒ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

   ____

   ____

   ____

   ____

   ____

   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☒ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☒ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ☒ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

☒ Yes ☐ No

If *Yes,* please list the consultant(s).

____

____

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 100 %
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

____

____

____

____

____

____

____

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ☒ No

If *Yes,* please briefly describe the problem(s).

____

____

____

____

____

____

____

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

[ ] Yes [x] No

If *Yes,* what design provisions are used to calculate the strength reduction?

[x] AASHTO LRFD Specifications [ ] AASHTO Segmental Bridge Specifications

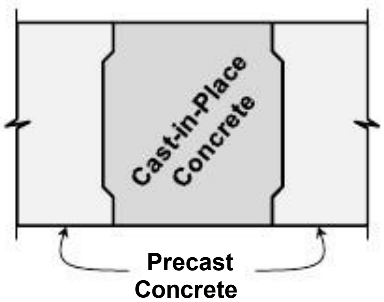
[ ] Other; please specify: ____________________

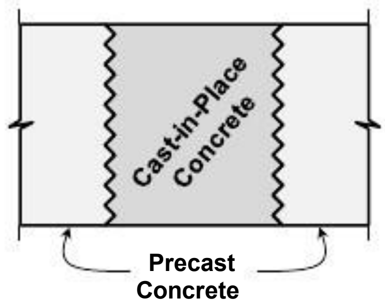
9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

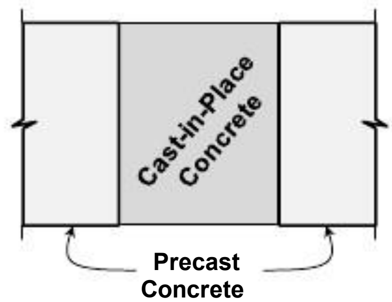
[ ] Yes [x] No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

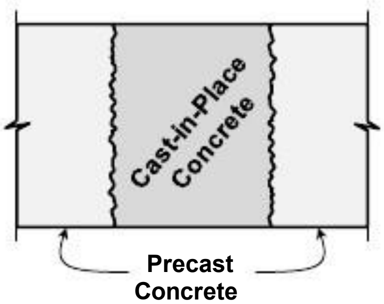
[ ] Shear key: ________% [x] Saw teeth: 100 % [ ] Plain: ________%







[ ] Sandblasting or intentional roughening: ________%

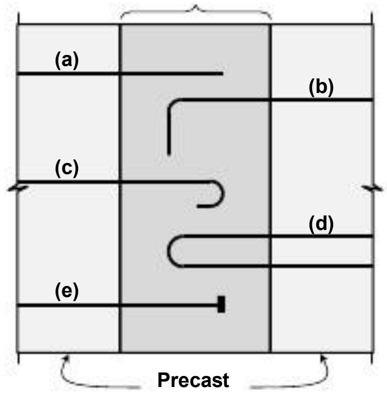


Please explain the factors that affect the type of interface that is chosen.

Complied with WSDOT Standard Details for projects that contained WSDOT Girders.

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

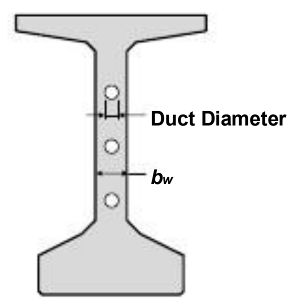
- [x] (a) Straight bars
- [ ] (b) 90-degree hooks
- [ ] (c) 180-degree hooks
- [ ] (d) Hairpins
- [ ] (e) Headed bars
- [ ] Other; please describe: ______________________



Please elaborate on the detailing of the interface reinforcement if necessary.

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
8 1/2"	4 5/8"	33 %
7 7/8"	4 3/8"	33 %
14"	4 3/8"	33 %
		%
		%

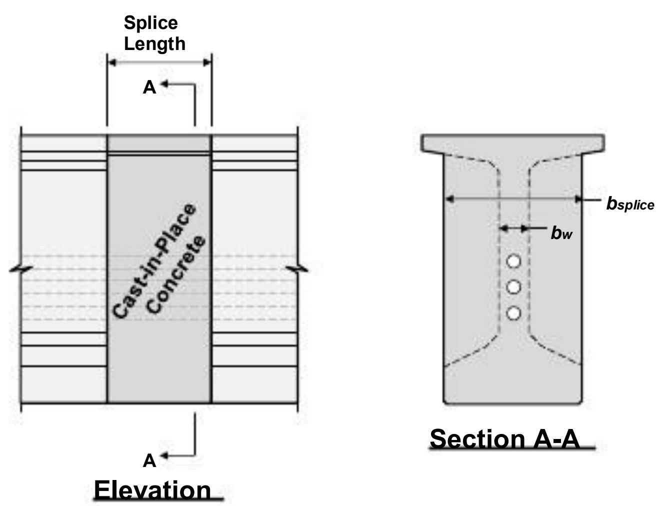


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

■ At the splice ☐ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	2 ft	3 ft	2 ft
$b_{splice} - b_w$			



Please explain the factors that affect the **length** of the splice region?

1) Complied with WSDOT Standard Details for projects that contained WSDOT Girders.

2) Constructability

Please explain the factors that affect the **width** of the splice region?

1) Complied with WSDOT Standard Details for projects that contained WSDOT Girders.

2) Constructability

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

We have only had three bridges utilizing spliced girders. Two were I- girders but one of them was a varying section (arched shape) rectangular section.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here: ______

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

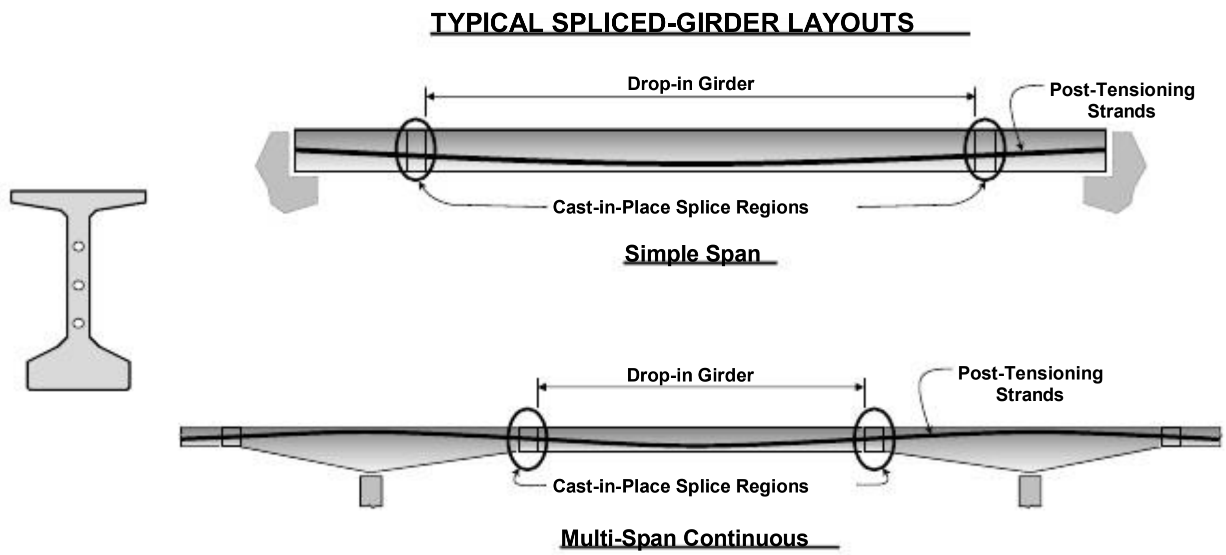
Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:	The University of Texas Research Team:
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Illinois

Organization/Unit: Bureau of Bridges and Structures

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☐ Yes ☒ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☒ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   We have not given serious consideration to the use of spliced girders.
   We have not used spliced girder technology and do not intend to, until such time the technology and practices are more widely accepted. I believe the Illinois Toll Highway Authority has used this technology, but they would have to be contacted separately.

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

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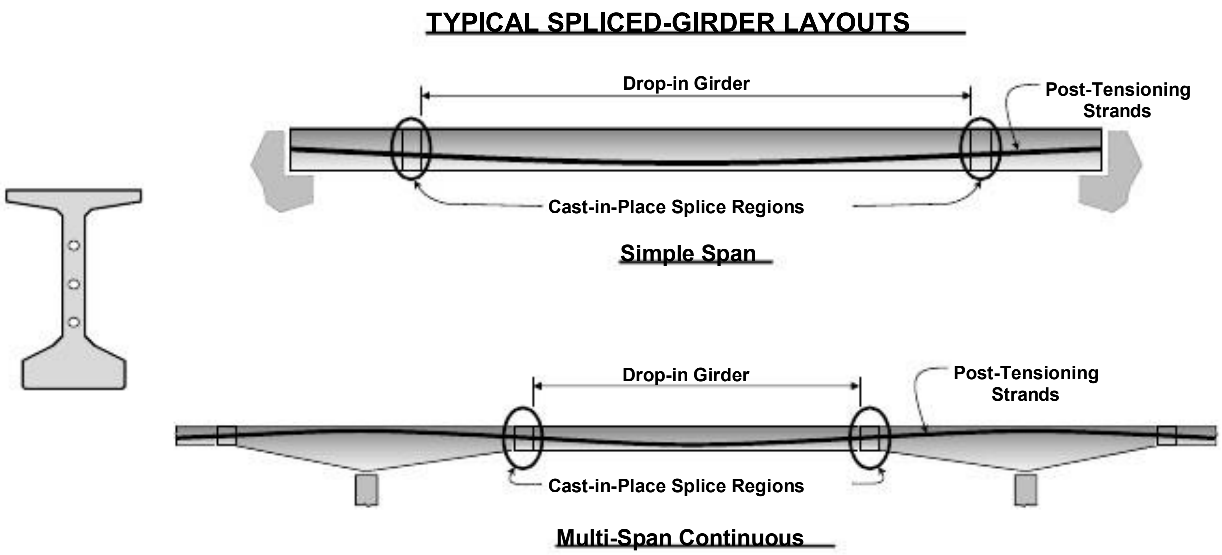
Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:	**The University of Texas Research Team:**
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ____________________

Title: ____________________

State/District: Iowa

Organization/Unit: Iowa Department of Transportation - Office of Bridges and Structures

Address: ____________________

____

Phone: ____________________

Fax: ____________________

Email: ____________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

☐ Yes ■ No

If *No,* has your state/district considered the use of spliced girder technology?

☐ Yes ■ No

If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

The need for temporary supports makes this technology less attractive when compared to traditional welded steel plate girder system with bolted field splices.

If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

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Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

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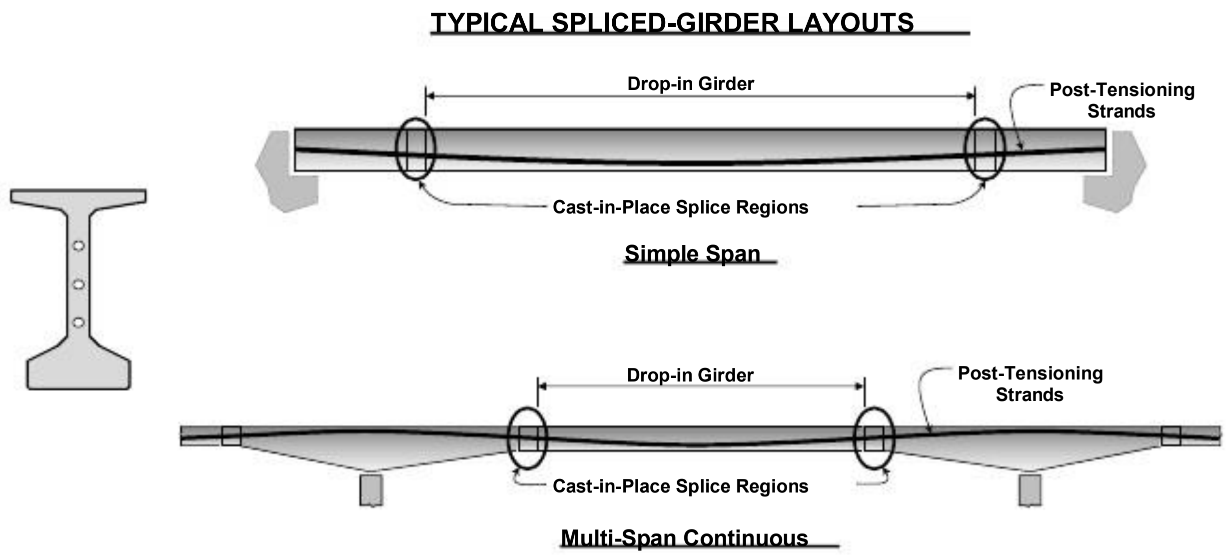

Texas Department of Transportation Contact:	**The University of Texas Research Team:**
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Kansas

Organization/Unit: Design

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

☐ Yes ☒ No

If *No,* has your state/district considered the use of spliced girder technology?

☒ Yes ☐ No

If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

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If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

☒ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

☒ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☒ Yes ☐ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

☒ Yes ☐ No

If *Yes,* please list the consultant(s).

____

____

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: ________%
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

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7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ☒ No

If *Yes,* please briefly describe the problem(s).

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8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

☐ Yes ☒ No

If *Yes,* what design provisions are used to calculate the strength reduction?

☐ AASHTO LRFD Specifications ☐ AASHTO Segmental Bridge Specifications

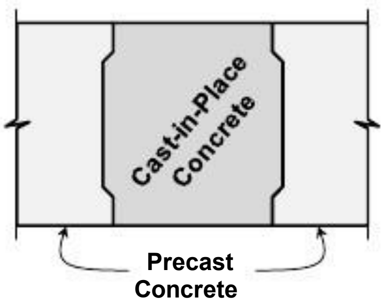
☐ Other; please specify: ____

9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

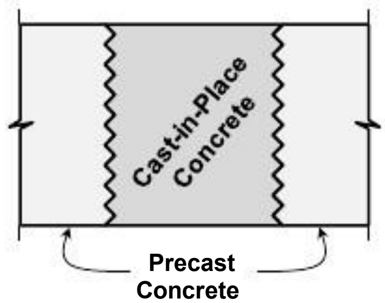
☐ Yes ☒ No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

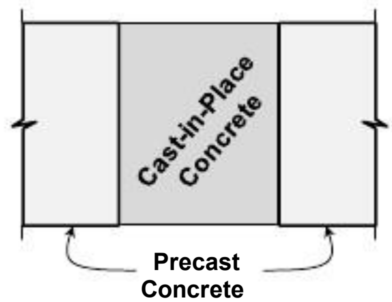
☐ Shear key: ________%



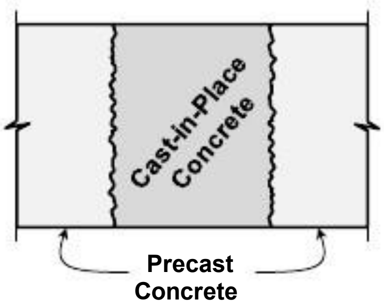
☐ Saw teeth: ________%



☐ Plain: ________%



☐ Sandblasting or intentional roughening: ________%



Please explain the factors that affect the type of interface that is chosen.

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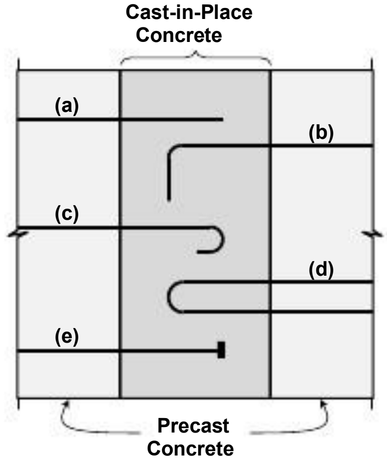
11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

- [ ] (a) Straight bars
- [x] (b) 90-degree hooks
- [ ] (c) 180-degree hooks
- [ ] (d) Hairpins
- [ ] (e) Headed bars
- [ ] Other; please describe: ______________________



Please elaborate on the detailing of the interface reinforcement if necessary.

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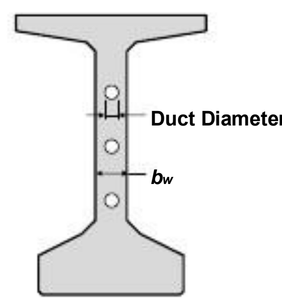
12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
		%
		%
		%
		%
		%

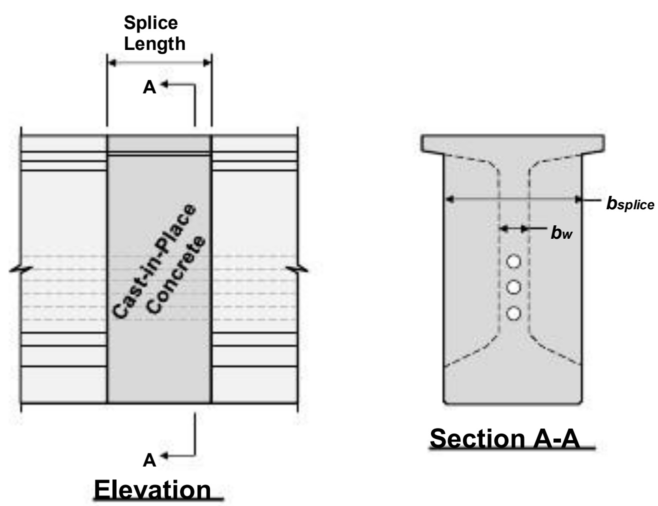


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

☐ At the splice ☐ Away from the splice ☒ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	**Minimum**	**Maximum**	**Typical**
Length			
$b_{splice} - b_w$			



Please explain the factors that affect the **length** of the splice region?

Please explain the factors that affect the **width** of the splice region?

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ■ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☐ Yes ■ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here: ______

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

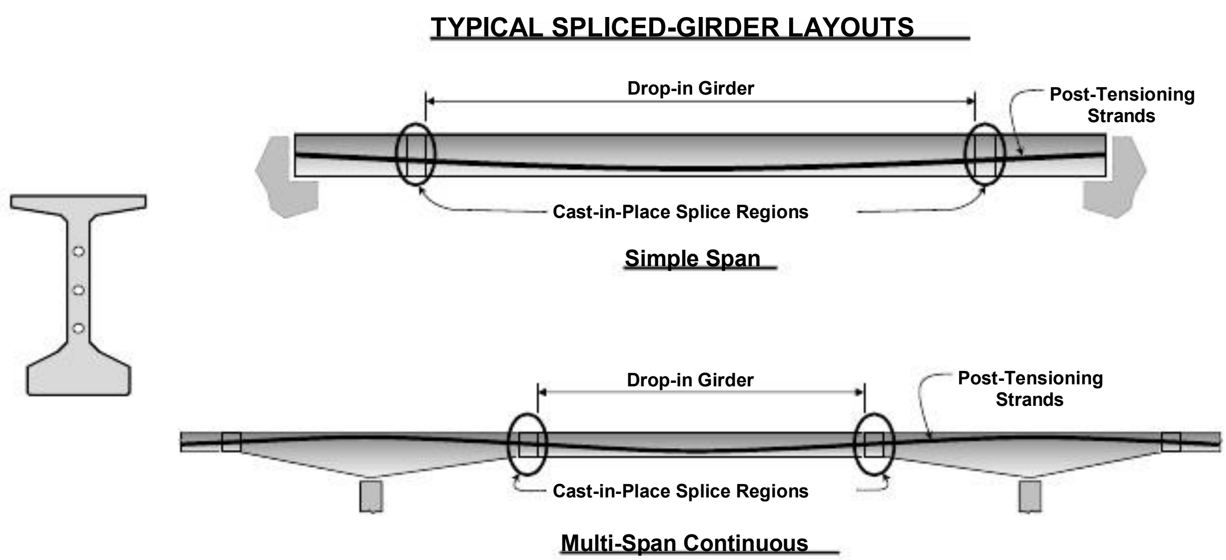
Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:	**The University of Texas Research Team:**
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Maryland

Organization/Unit: Maryland State Highway Administration / Office of Structures

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☐ Yes ☒ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☒ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   We are not comfortable with this technology. For large spans that would require splicing of concrete girders, we would use steel girders.

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

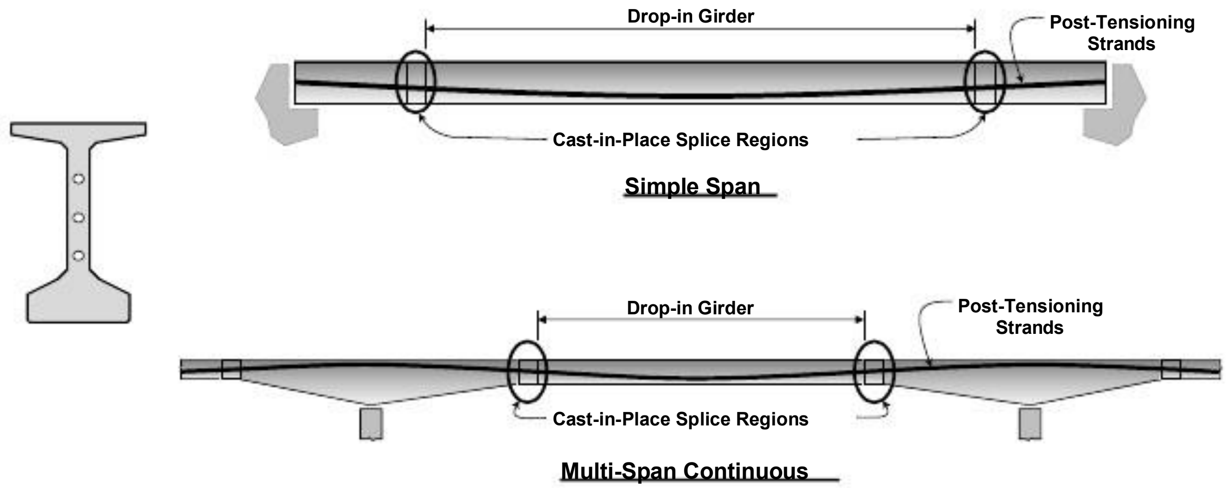
Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division

TYPICAL SPLICED-GIRDER LAYOUTS



Texas Department of Transportation Contact:
Greg Turco, PE

Address:
Bridge Division

The University of Texas Research Team:
Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu
Dr. James Jirsa: jirsa@uts.cc.utexas.edu
Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu
Chris Williams: chrisw05@utexas.edu
Andy Moore: ammoore@utexas.edu

Address:
Ferguson Structural Engineering Laboratory
The University of Texas at Austin
10100 Burnet Rd., Building 177
Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: MA

Organization/Unit: MassDOT / Highway Division / Bridge Section

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☒ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

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   ____

   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☒ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☒ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☒ Yes ☐ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

☒ Yes ☐ No

If *Yes,* please list the consultant(s).

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 100 %
- Plastic (HDPE): 0 %

If one of the materials is preferred over the other, please explain why.

Steel is considered standard, therefor preferred by MassDOT.

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ☒ No

If *Yes,* please briefly describe the problem(s).

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

☐ Yes ☒ No

If *Yes,* what design provisions are used to calculate the strength reduction?

☐ AASHTO LRFD Specifications ☐ AASHTO Segmental Bridge Specifications

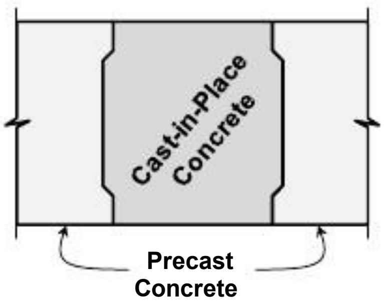
☐ Other; please specify: ______________________________

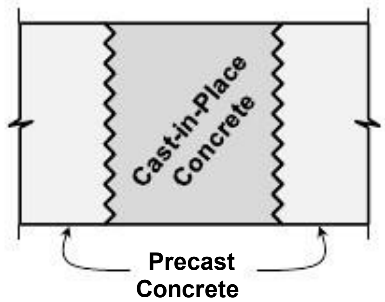
9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

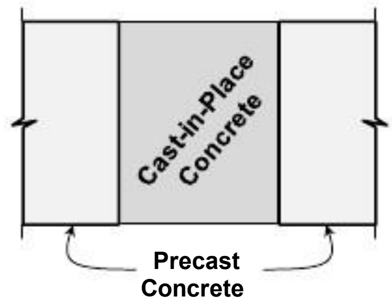
☐ Yes ☒ No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

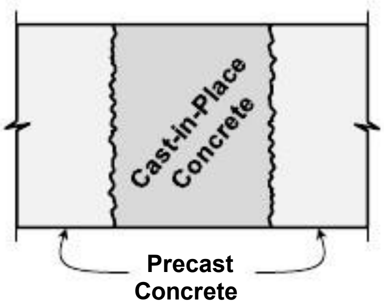
☒ Shear key: 100 % ☐ Saw teeth: ________% ☐ Plain: ________%







☐ Sandblasting or intentional roughening: ________%



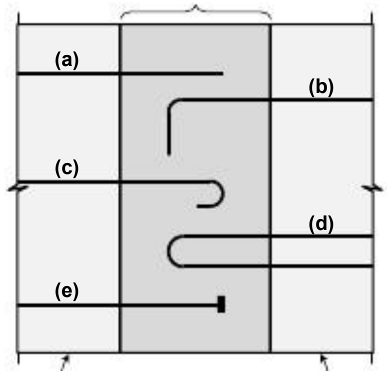
Please explain the factors that affect the type of interface that is chosen.

Interface: 3 - 4" x 10" recessed keys @ 2" deep (1 in web, 2 in bulb)

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

☐ (a) Straight bars ☐ (b) 90-degree hooks ☐ (c) 180-degree hooks ■ (d) Hairpins

☐ (e) Headed bars ☐ Other; please describe: ____________________



Please elaborate on the detailing of the interface reinforcement if necessary.

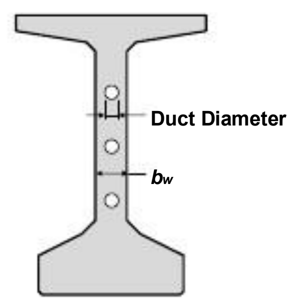
Splice with Transverse Diaphragms at the Splice location.

Interior Splice: #4 (d) Typ. with 4 - #4 @ 6'-0" Long Centered on Girder

Exterior Splice: #4 (d) Typ. with 1 - #4 @ 5'-0" Long at Top Frame and 3 - #4 (c) at Webs and Bulb

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
7"	3"	100 %
		%
		%
		%
		%

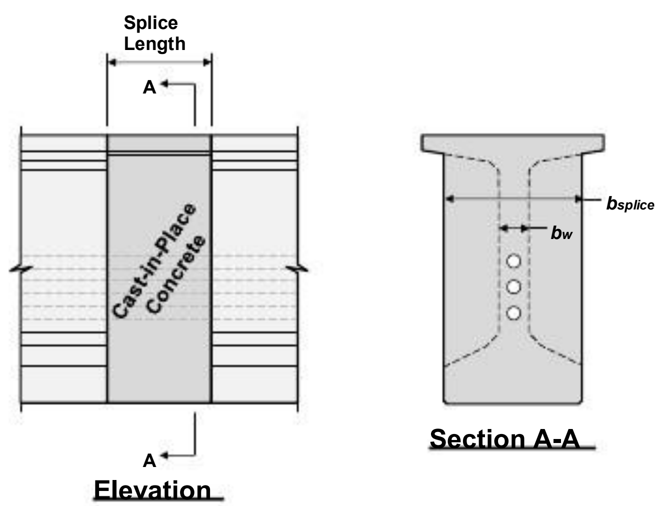


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

■ At the splice ☐ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., b_{splice} - b_w), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the (b_{splice} - b_w) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	12"	14"	12"
b_{splice} - b_w	-	-	-



Please explain the factors that affect the **length** of the splice region?

PCI guidelines on spliced girders recommended a splice length of 250mm, therefor 12" was used.

Please explain the factors that affect the **width** of the splice region?

N/A

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☒ Yes ☐ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

We were made aware of issues in the splice region during construction including misalignment and damage to ducts. A repair procedure for the duct was developed by inserting a smaller duct inside the existing duct. A mock up was created and splices were implemented in field.

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

See relevant drawing sheets which have been uploaded to your FTP site.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here: ______

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

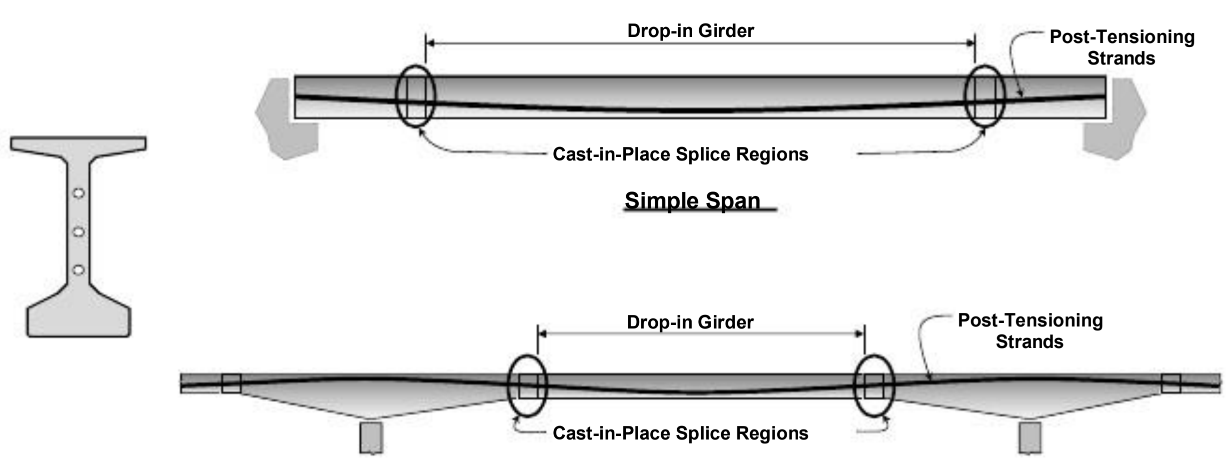
Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division

TYPICAL SPLICED-GIRDER LAYOUTS



Texas Department of Transportation Contact:	The University of Texas Research Team:
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: __________

Title: __________

State/District: Michigan DOT/Central Office

Organization/Unit: Bureau of Field Services

Address: __________

Phone: __________

Fax: __________

Email: __________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

☐ Yes ☒ No

If *No,* has your state/district considered the use of spliced girder technology?

☒ Yes ☐ No

If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

We are considering it, however, we are still looking for the right location and application.
MDOT would be very interested in the research results.

If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

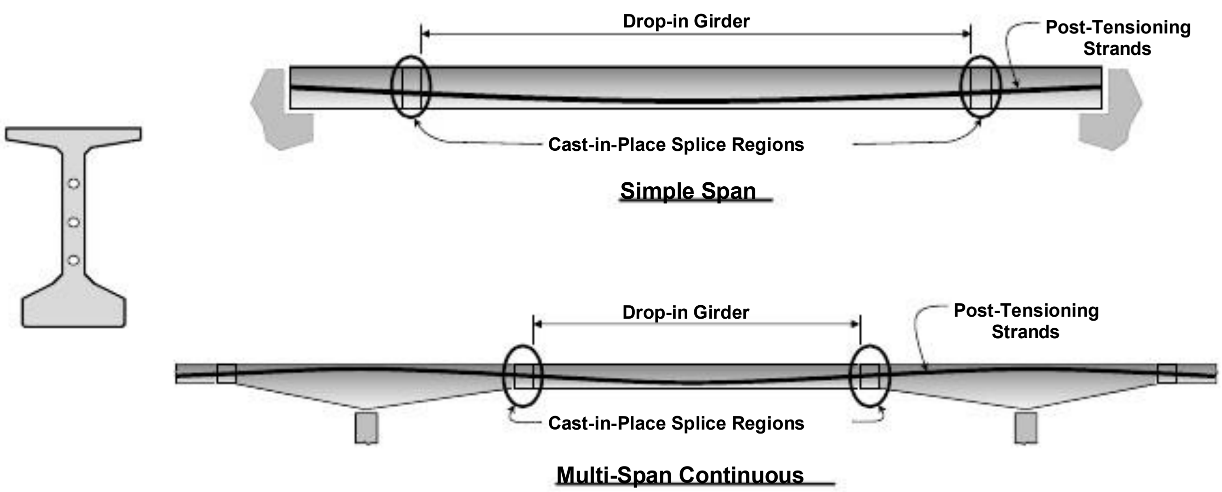
<u>Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region</u>

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division

TYPICAL SPLICED-GIRDER LAYOUTS



Texas Department of Transportation Contact:	**The University of Texas Research Team:**
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______

Title: ______

State/District: Minnesota

Organization/Unit: MnDOT - Bridge Office

Address: ______

Phone: ______

Fax: ______

Email: ______

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☒ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   MnDOT has constructed 2 spliced prestressed girder bridges back in 1993. No other spliced girder bridges been completed since that time, so only Part A of this survey has been completed as our experience on this is not current. I will, however, provide pertinent sheets from the bridge plans (see attached file: br 70037 and 70038.pdf)

   MnDOT does consider spliced girders as an option for new bridges.

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☒ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☒ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ■ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

■ Yes ☐ No

If *Yes,* please list the consultant(s).

____

____

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: ________%
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

____

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____

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ☐ No

If *Yes,* please briefly describe the problem(s).

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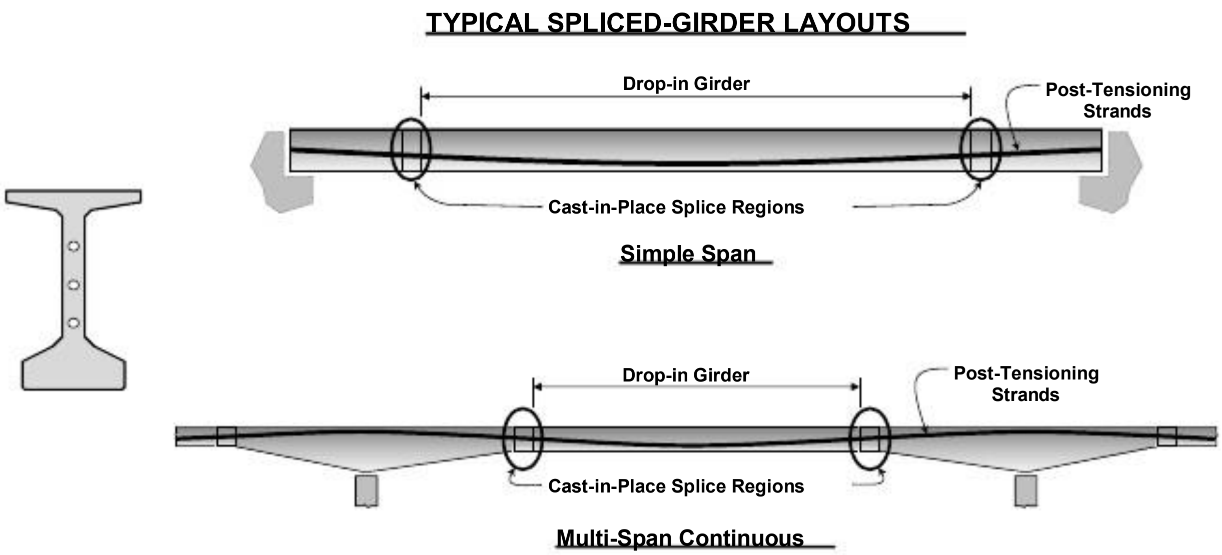
Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:
Greg Turco, PE

Address:
Bridge Division

The University of Texas Research Team:
Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu
Dr. James Jirsa: jirsa@uts.cc.utexas.edu
Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu
Chris Williams: chrisw05@utexas.edu
Andy Moore: ammoore@utexas.edu

Address:
Ferguson Structural Engineering Laboratory
The University of Texas at Austin
10100 Burnet Rd., Building 177
Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______

Title: ______

State/District: Missouri

Organization/Unit: MoDOT (Bridge Division)

Address: ______

______

Phone: ______

Fax: ______

Email: ______

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

☐ Yes ☒ No

If *No,* has your state/district considered the use of spliced girder technology?

☒ Yes ☐ No

If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

Spliced girder technology was used on one, maybe two bridges in the past. However, that was was over 20 years ago, and we don't have technical expertise in this area to fill out the rest of the survey.

If spliced girder technology is reliable and cost effective then MoDOT may consider for long span bridges.

If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

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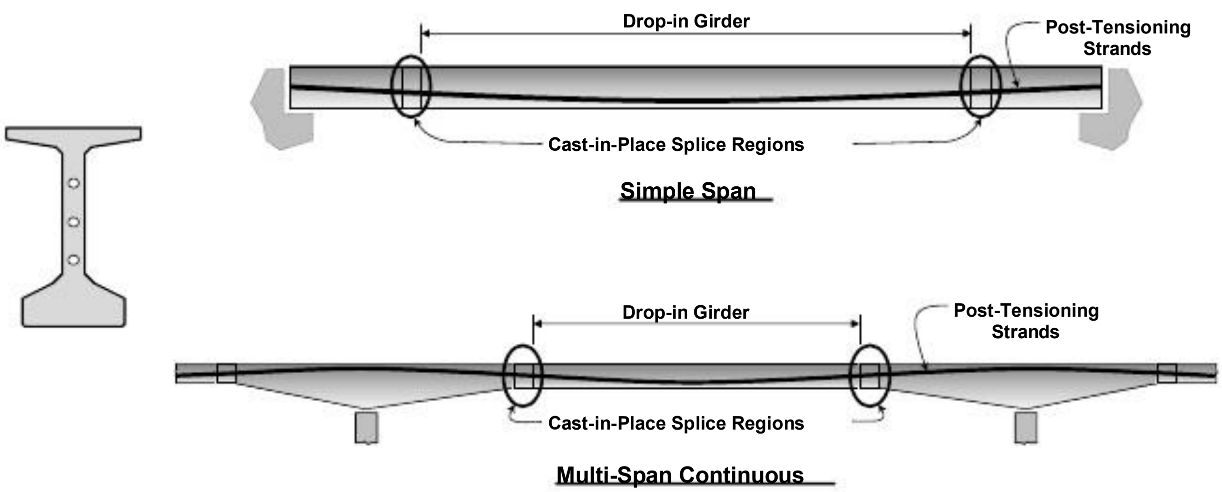
TYPICAL SPLICED-GIRDER LAYOUTS



Texas Department of Transportation Contact: Greg Turco, PE **Address:** Bridge Division	**The University of Texas Research Team:** Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Montana

Organization/Unit: Montana Department of Transportation Bridge Bureau

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☐ Yes ■ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ■ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

   ____

   ____

   ____

   ____

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   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

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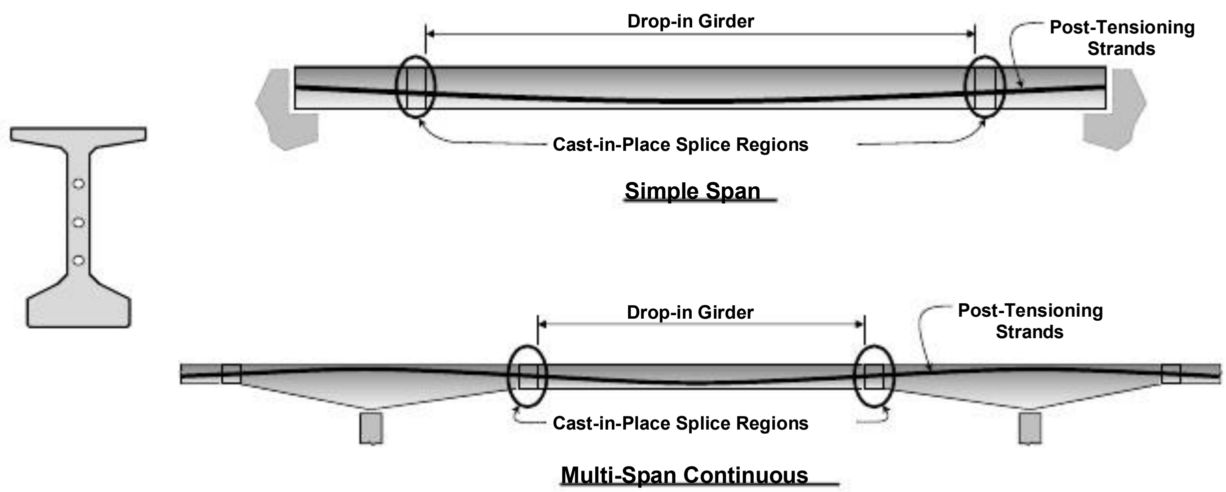


Texas Department of Transportation Contact:	**The University of Texas Research Team:**
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Nevada

Organization/Unit: Department of Transportation

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ■ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   Precast girders are not typically cost competitive as no PCI certified precaster is located within the state.
   Contractor's may be allowed to site-cast the precast boxes and U's for bridge widenings, if the girder geometry is simple (i.e. little or no skew, no curvature). These type of girders were used on several projects from 1995-2000.
   We have also widened a couple of structures in the past 2-3 years with this method.

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ■ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ■ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

- [x] Yes
- [ ] No

5. Has your state used consulting engineers for the design of spliced girder bridges?

- [x] Yes
- [ ] No

If *Yes,* please list the consultant(s).

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 100 %
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

Steel has been used for our box girder and u-girder bridges 100% of the time. As noted in Question #2, no spliced I girders have been used in Nevada.

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

- [ ] Yes
- [x] No

If *Yes,* please briefly describe the problem(s).

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

☐ Yes ■ No

If *Yes,* what design provisions are used to calculate the strength reduction?

☐ AASHTO LRFD Specifications ☐ AASHTO Segmental Bridge Specifications

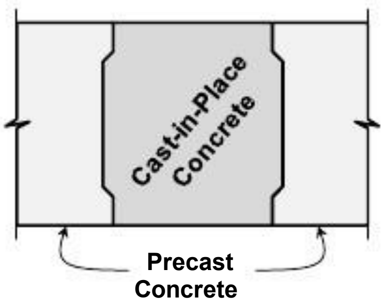
☐ Other; please specify: ____________________

9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

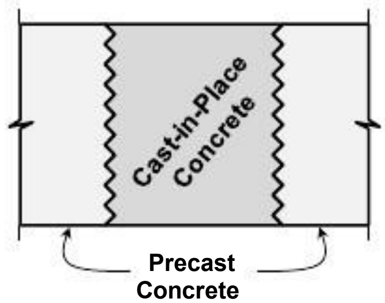
☐ Yes ■ No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

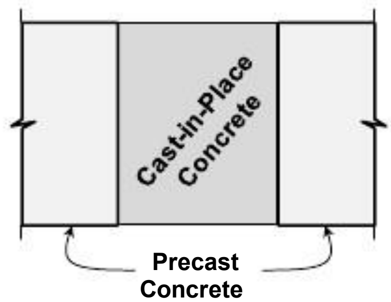
☐ Shear key: ________%



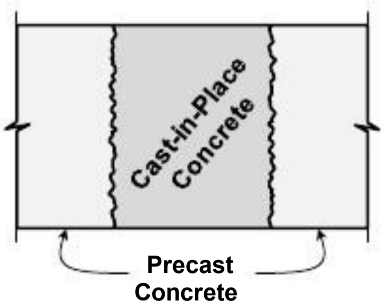
■ Saw teeth: 100 %



☐ Plain: ________%



☐ Sandblasting or intentional roughening: ________%

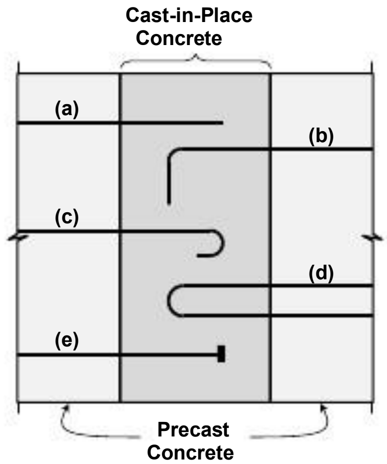


Please explain the factors that affect the type of interface that is chosen.

Saw tooth detail has been standard practice.

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

- [x] (a) Straight bars
- [ ] (b) 90-degree hooks
- [ ] (c) 180-degree hooks
- [ ] (d) Hairpins
- [ ] (e) Headed bars
- [ ] Other; please describe: ______________________



Please elaborate on the detailing of the interface reinforcement if necessary.

____

____

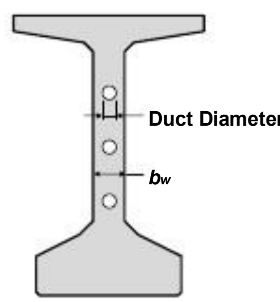
____

____

____

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	**Duct Diameter**	**Percent of Projects**
		%
		%
		%
		%
		%

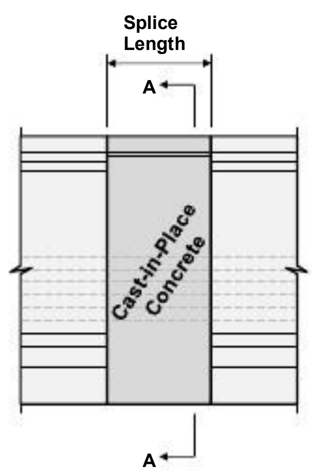


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

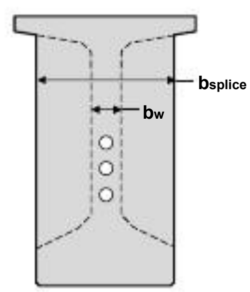
☐ At the splice ☐ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length			
$b_{splice} - b_w$			



Elevation



Section A-A

Please explain the factors that affect the **length** of the splice region?

Typically, it is preferred to match existing pier diaphragm length and width if possible.

Please explain the factors that affect the **width** of the splice region?

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☒ Yes ☐ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

Potential rebar conflicts with vertical steel with columns (particularly hooks) and horizontal steel from girder webs needs to be addressed. A threading detail should be included if necessary.

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here:

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

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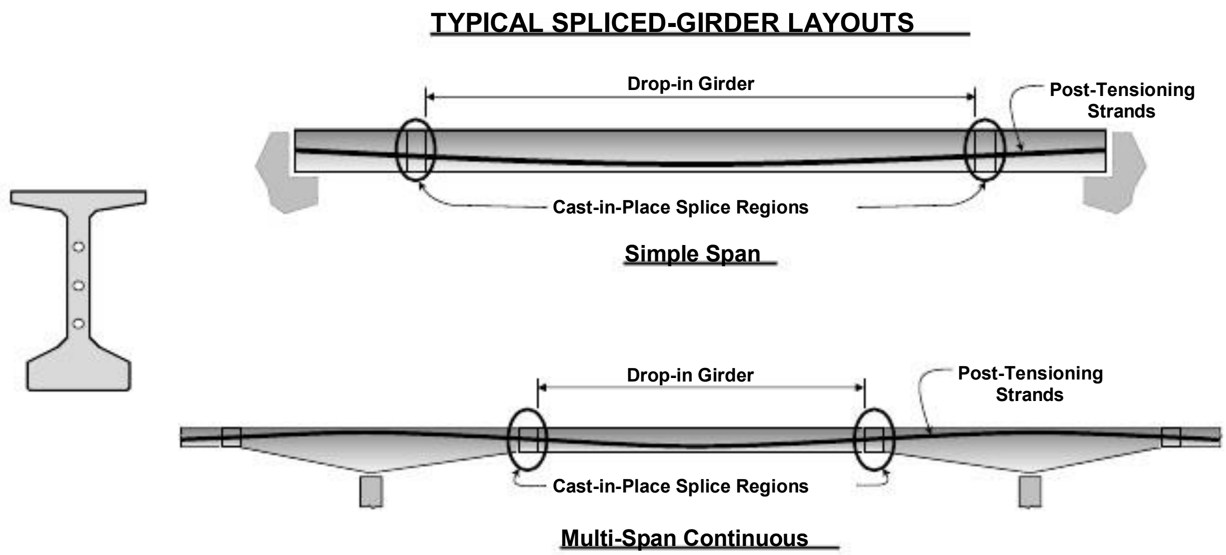

Texas Department of Transportation Contact:	The University of Texas Research Team:
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: New York State Department of Transportation

Organization/Unit: Office of Structures

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☒ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

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   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☒ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☒ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

■ Yes ☐ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

☐ Yes ■ No

If *Yes,* please list the consultant(s).

____

____

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 100 %
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

____

____

____

____

____

____

____

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ■ No

If *Yes,* please briefly describe the problem(s).

____

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____

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

☐ Yes ☒ No

If *Yes,* what design provisions are used to calculate the strength reduction?

☐ AASHTO LRFD Specifications ☐ AASHTO Segmental Bridge Specifications

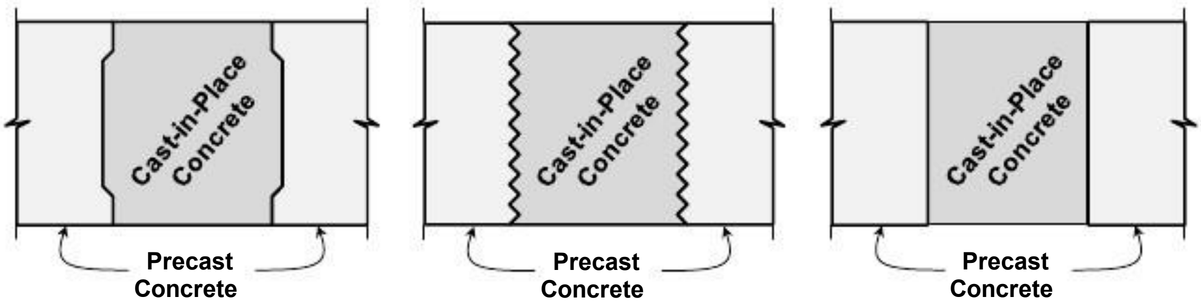
☐ Other; please specify: ______________________________

9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

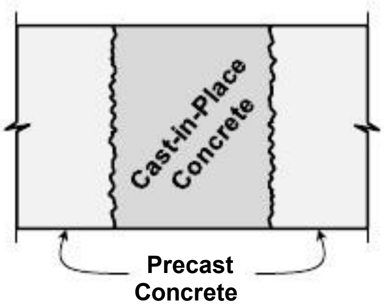
☐ Yes ☒ No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

☒ Shear key: 100 % ☐ Saw teeth: ________% ☐ Plain: ________%



☐ Sandblasting or intentional roughening: ________%

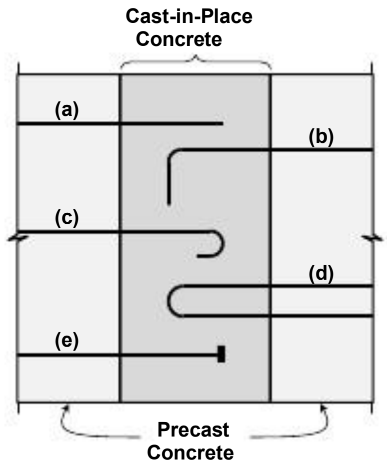


Please explain the factors that affect the type of interface that is chosen.

We believe a shear key provides the best shear transfer mechanism.

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

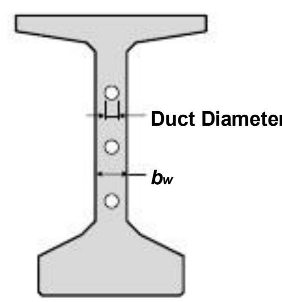
- [ ] (a) Straight bars
- [x] (b) 90-degree hooks
- [ ] (c) 180-degree hooks
- [x] (d) Hairpins
- [ ] (e) Headed bars
- [ ] Other; please describe: ____________________



Please elaborate on the detailing of the interface reinforcement if necessary.

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
8	4	50 %
7	3	50 %
		%
		%
		%

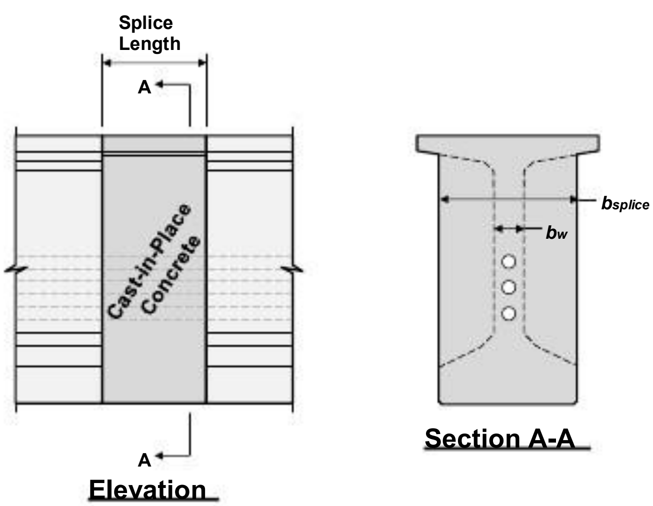


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

■ At the splice ☐ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	10"	None	10"
$b_{splice} - b_w$	-	-	-



Please explain the factors that affect the **length** of the splice region?

Need enough room for the concrete to flow around the ducts.

Please explain the factors that affect the **width** of the splice region?

Aesthetics.

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☒ Yes ☐ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

Splice concrete color normally does not match the color of the girder.

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

■ Yes ☐ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

We have had issues with splice concrete not flowing properly resulting in voids and honeycombing. Splices had to be removed and re-poured.

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

Contract plans and shop drawings will be uploaded to the Texas DOT dropbox.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here: None

Any other relevant information you can offer the research team will be greatly appreciated.

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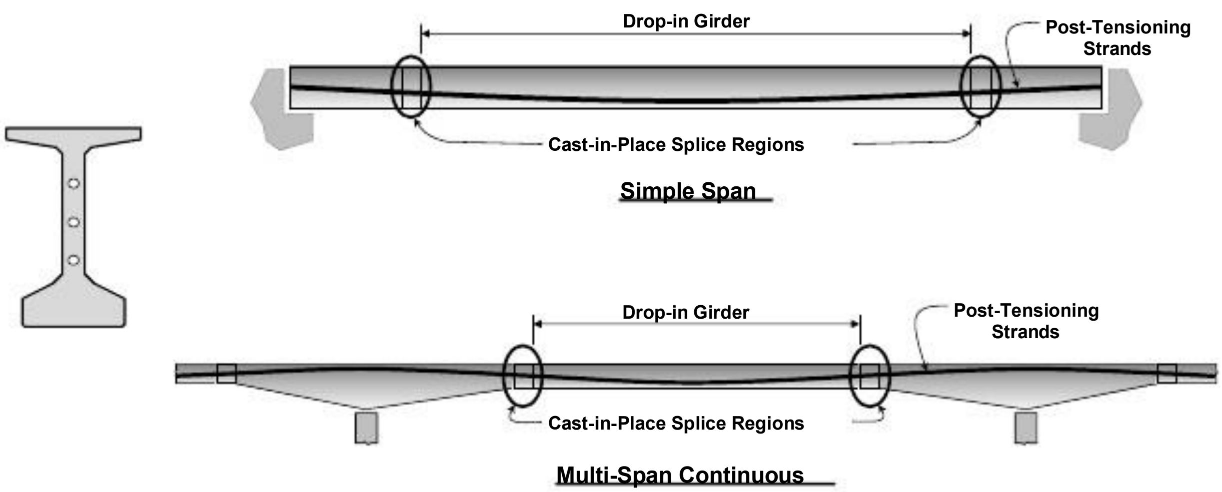
TYPICAL SPLICED-GIRDER LAYOUTS



Texas Department of Transportation Contact:
Greg Turco, PE

Address:
Bridge Division

The University of Texas Research Team:
Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu
Dr. James Jirsa: jirsa@uts.cc.utexas.edu
Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu
Chris Williams: chrisw05@utexas.edu
Andy Moore: ammoore@utexas.edu

Address:
Ferguson Structural Engineering Laboratory
The University of Texas at Austin
10100 Burnet Rd., Building 177
Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: North Carolina

Organization/Unit: NCDOT/Structures Management Unit

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ■ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

   ____

   ____

   ____

   ____

   ____

   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ■ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ■ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ☒ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

☒ Yes ☐ No

If *Yes,* please list the consultant(s).

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 100 %
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

When comparing plastic ducts to galvanized corrugated metal ducts during the design/plan development stage, NCHRP Report 517 served as a reference. The report noted that metal ducts required less support to prevent misalignment and displacement during the casting of the girder.

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ☒ No

If *Yes,* please briefly describe the problem(s).

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

☐ Yes ☒ No

If *Yes,* what design provisions are used to calculate the strength reduction?

☐ AASHTO LRFD Specifications ☐ AASHTO Segmental Bridge Specifications

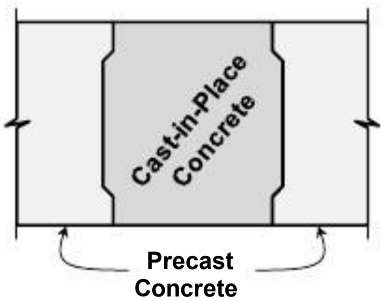
☐ Other; please specify: ______________________________

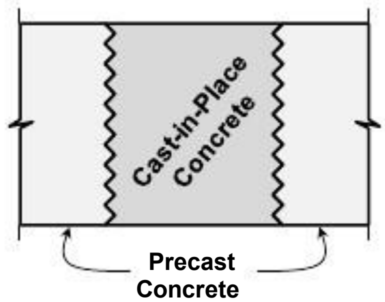
9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

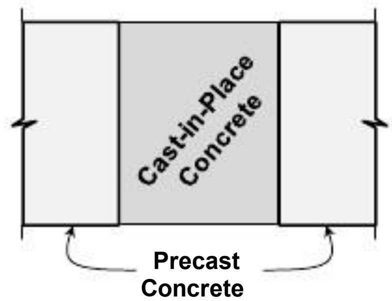
☐ Yes ☒ No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

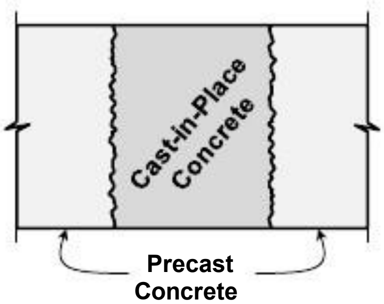
☒ Shear key: 100 % ☐ Saw teeth: ________% ☐ Plain: ________%







☐ Sandblasting or intentional roughening: ________%



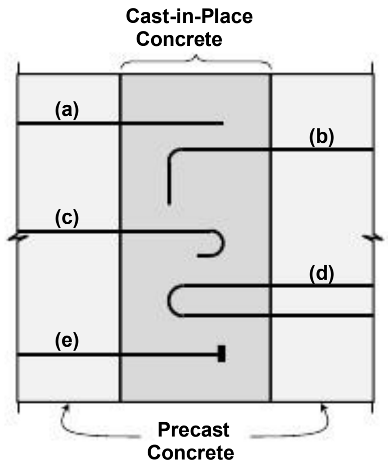
Please explain the factors that affect the type of interface that is chosen.

Simple detail that is easy to fabricate and control during fabrication

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

☐ (a) Straight bars ☒ (b) 90-degree hooks ☒ (c) 180-degree hooks ☒ (d) Hairpins

☐ (e) Headed bars ☐ Other; please describe: ____________________

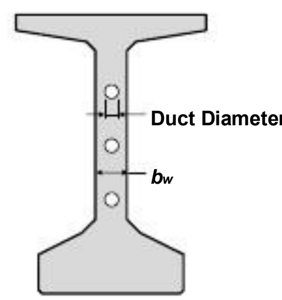


Please elaborate on the detailing of the interface reinforcement if necessary.

90-degree hooks are detailed in the web of the girder; 180-degree hooks are detailed in the top flange of the girder; Hairpins are detailed in the bulb of the girder.

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
9"	3.82"	67 %
8"	3.42"	33 %
		%
		%
		%

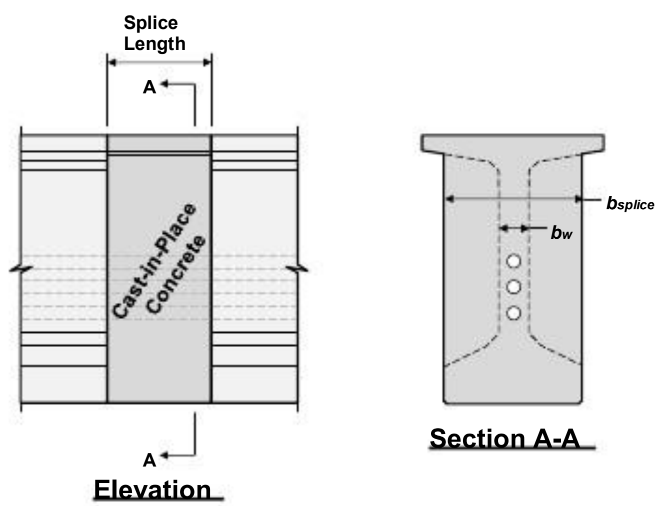


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

■ At the splice ☐ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	2'-0"	Dependent upon skew	2'-0"
$b_{splice} - b_w$			



Please explain the factors that affect the **length** of the splice region?

The length should provide an adequate opening for proper placement of cast-in-place concrete.

Please explain the factors that affect the **width** of the splice region?

Transverse diaphragms are generally located at a splice location; therefore, the width is dependent upon the typical section.

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ■ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☒ Yes ☐ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

Strongback failure during girder erection. Spliced girder designs and falsework & formwork submittals are reviewed by an independent third party.

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here: ______

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

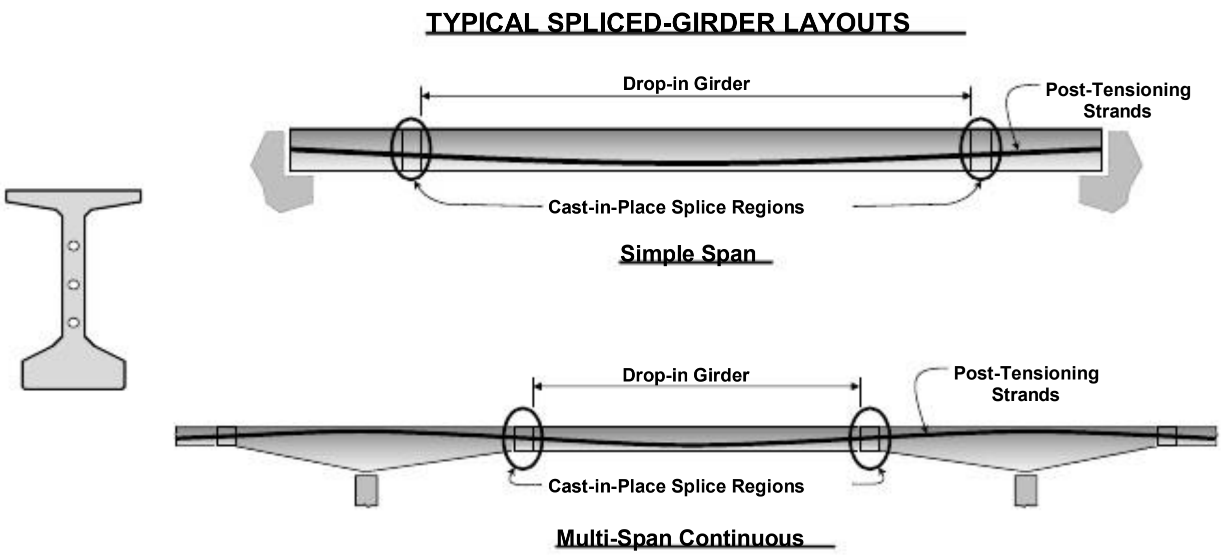
Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:	**The University of Texas Research Team:**
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Ohio

Organization/Unit: Department of Transportation

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☐ Yes ■ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ■ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   There have been 1-2 local projects that have utilized spliced I-beam designs and one contractor value engineering proposal that changed the original design to a spliced I-beam design. This is not a structure type that ODOT prefers due to the lack of ability to inspect the major load carrying component - PT Tendon.

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ■ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ■ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ■ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

■ Yes ☐ No

If *Yes,* please list the consultant(s).

____

____

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: ________%
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

____

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7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ☐ No

If *Yes,* please briefly describe the problem(s).

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Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

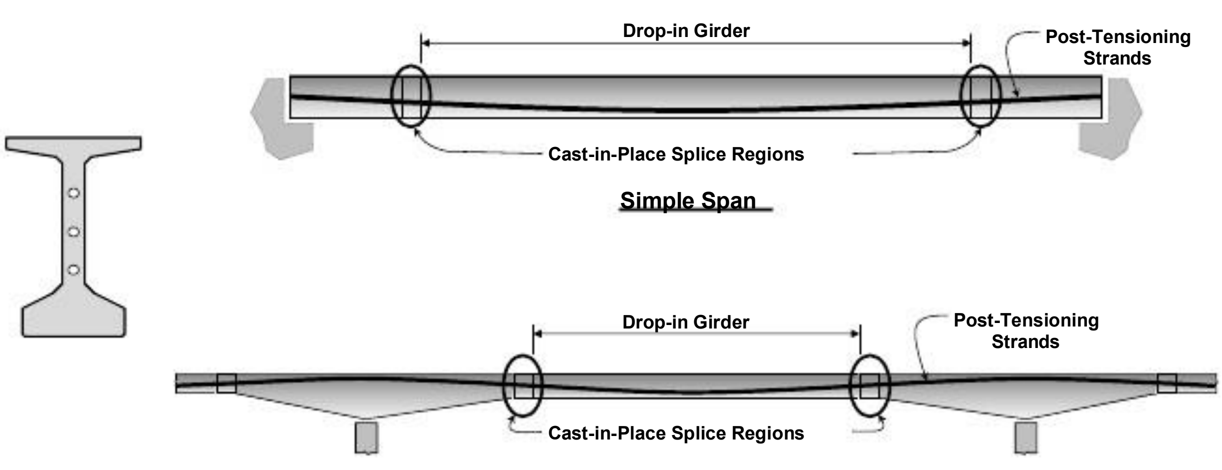
Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division

TYPICAL SPLICED-GIRDER LAYOUTS



Texas Department of Transportation Contact: Greg Turco, PE **Address:** Bridge Division	**The University of Texas Research Team:** Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Pennsylvania Department of Transportation

Organization/Unit: Bridge Design and Technology Division

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

☐ Yes ■ No

If *No,* has your state/district considered the use of spliced girder technology?

■ Yes ☐ No

If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

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If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

■ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

■ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ■ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

■ Yes ☐ No

If *Yes,* please list the consultant(s).

Pennsylvania will use a consultant to perform the girder design and a have a second consultant perform a peer review of the design for the first project to utilize spliced girders.

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: n/a %
- Plastic (HDPE): n/a %

If one of the materials is preferred over the other, please explain why.

Pennsylvania permits plastic ducts, if the tendon radius is greater than 30-feet (A5.4.6.1) and require galvanized steel ducts for smaller tendon radius.

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ■ No

If *Yes,* please briefly describe the problem(s).

Indicated 'NO' since no project completed in Pennsylvania at this time.

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

☒ Yes ☐ No

If *Yes,* what design provisions are used to calculate the strength reduction?

☒ AASHTO LRFD Specifications ☐ AASHTO Segmental Bridge Specifications

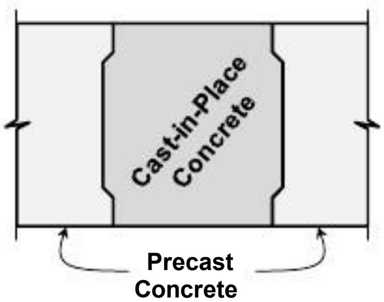
☐ Other; please specify: ____________________

9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

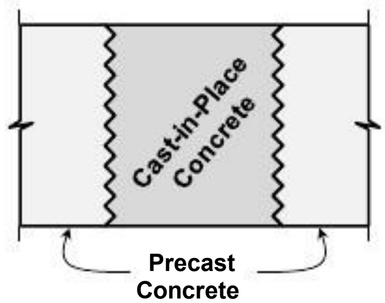
☐ Yes ☒ No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

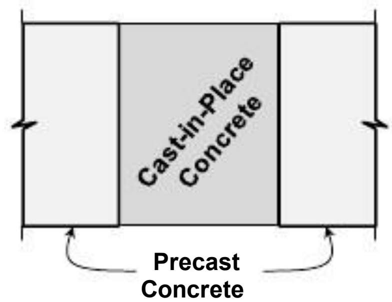
☐ Shear key: ________%



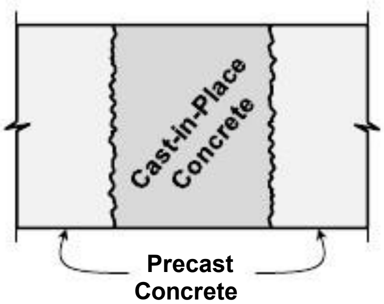
☒ Saw teeth: n/a %



☐ Plain: ________%



☐ Sandblasting or intentional roughening: ________%



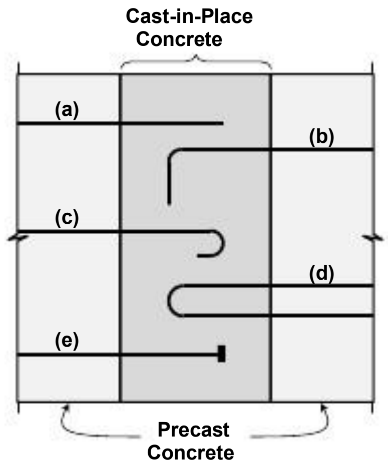
Please explain the factors that affect the type of interface that is chosen.

The saw tooth beam end treatment is specified on the standard drawings developed for spliced girder projects in Pennsylvania.

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

☐ (a) Straight bars ☒ (b) 90-degree hooks ☐ (c) 180-degree hooks ☒ (d) Hairpins

☐ (e) Headed bars ☐ Other; please describe: ______

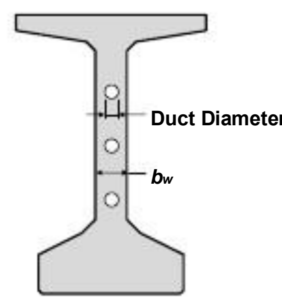


Please elaborate on the detailing of the interface reinforcement if necessary.

The standard drawings for simple span spliced girders utilizes a 90-degree hooked bar (b), that is bent in the field, in the top flange at the CIP splice. All other projecting reinforcement is detailed as hairpin bars (d). All projecting reinforcement for the continuous spliced girderstandards is detailed as hairpin bars (d).

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
8"	3.25"	n/a %
		%
		%
		%
		%

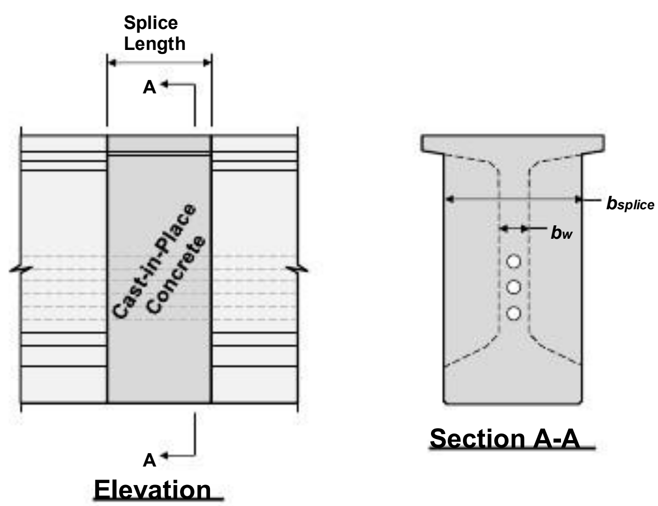


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

☐ At the splice ☒ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	1'-6"	1'-6"	1'-6"
$b_{splice} - b_w$	0	0	0



Please explain the factors that affect the **length** of the splice region?

An alternate detail for a keyed joint (shear key in question #10 above) indicates 1'-0" minimum at the CIP splice with 3" deep keys on each beam end (i.e. 1'-6" between beam ends within key).

Please explain the factors that affect the **width** of the splice region?

The CIP splice is formed to match the beam dimensions (i.e. CIP web width is 8" to match precast web width). Only the ends of the structure where the post-tensioning anchors are located have increased web widths. For simple span spliced girders, the maximum web width at the anchorage is 2'-0". For continuous spliced girders, the maximum web width at the anchorage is 2'-9" (equal to bottom flange width).

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

Indicated 'NO' since no project completed in Pennsylvania at this time.

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☐ Yes ■ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

Indicated 'NO' since no project completed in Pennsylvania at this time.

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here:

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

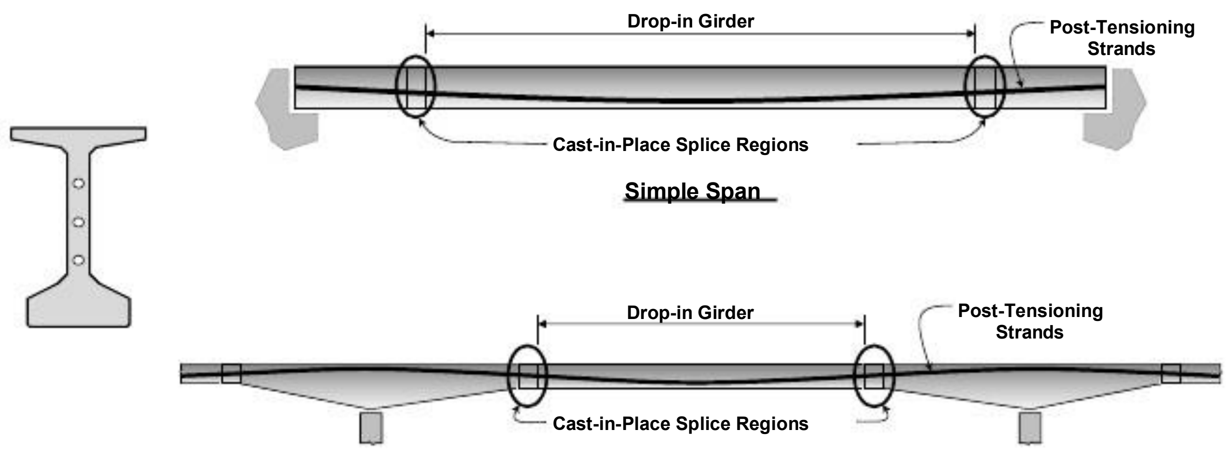
Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division

TYPICAL SPLICED-GIRDER LAYOUTS



Texas Department of Transportation Contact:	**The University of Texas Research Team:**
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: South Dakota

Organization/Unit: South Dakota DOT

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☐ Yes ☒ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☒ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

   ____

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   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

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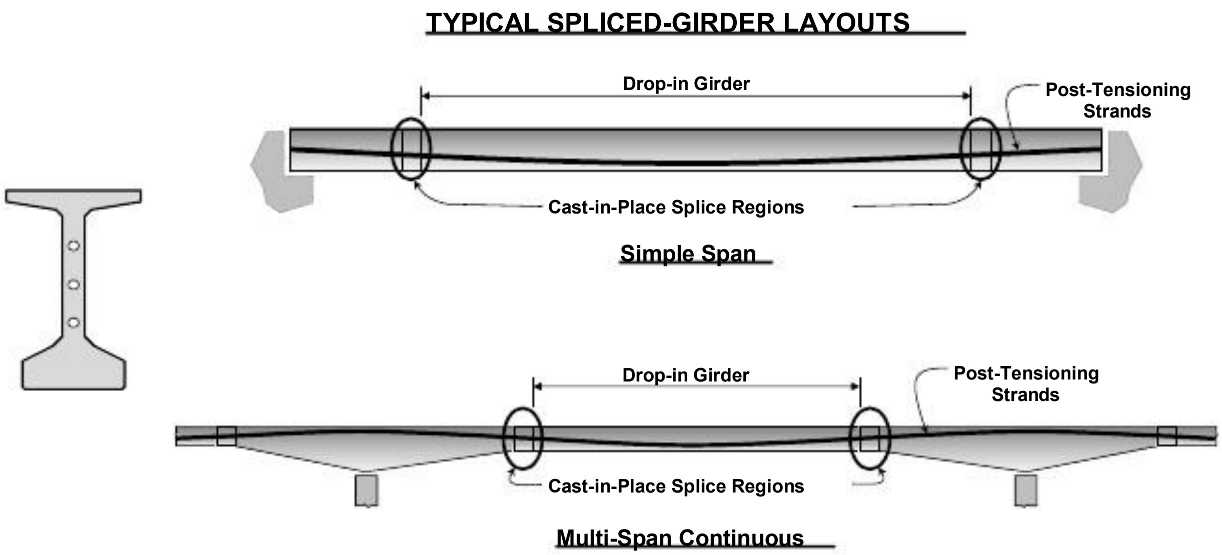
Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:	The University of Texas Research Team:
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ____________________

Title: ____________________

State/District: Vermont Agency of Transportation

Organization/Unit: Structures Section

Address: ____________________

Phone: ____________________

Fax: ____________________

Email: ____________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☐ Yes ☒ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☒ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   Vermont has typically chosen steel as the material for bridge spans where spliced prestressed girders would be applicable. We have recently allowed this as an option on some design/build projects.

   We have had a comfort level with steel and just have not tried this approach with traditional projects.

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

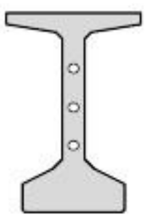
Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

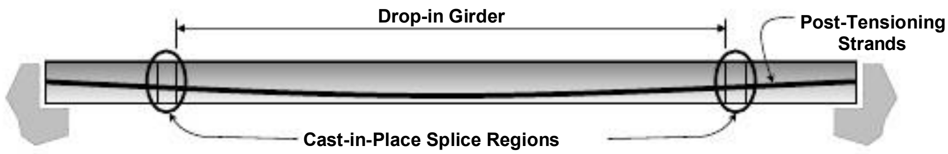
The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

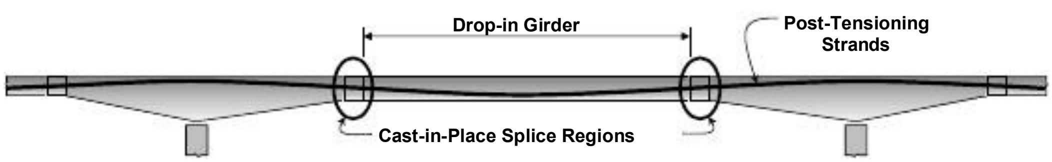
Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division

TYPICAL SPLICED-GIRDER LAYOUTS





Simple Span



Multi-Span Continuous

Texas Department of Transportation Contact:
Greg Turco, PE

Address:
Bridge Division

The University of Texas Research Team:
Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu
Dr. James Jirsa: jirsa@uts.cc.utexas.edu
Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu
Chris Williams: chrisw05@utexas.edu
Andy Moore: ammoore@utexas.edu

Address:
Ferguson Structural Engineering Laboratory
The University of Texas at Austin
10100 Burnet Rd., Building 177
Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: VIRGINIA

Organization/Unit: DEPARTMENT OF TRANSPORTATION - STRUCTURE & BRIDGE DIVISION

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ■ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

   ____

   ____

   ____

   ____

   ____

   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ■ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ■ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

☐ Yes ■ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

■ Yes ☐ No

If *Yes,* please list the consultant(s).

____

____

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: ________%
- Plastic (HDPE): ________%

If one of the materials is preferred over the other, please explain why.

HDPE is perferred because of better durability and smaller chance of damage during construction; however, on one project

Contractor sub- stitute polypropylene ducts w/o approval prior to fabrication of girder segments.

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

■ Yes ☐ No

If *Yes,* please briefly describe the problem(s).

Problems with sealing of ducts allowed grout to transmit into an adjacent empty duct (prior to tendon placement).

Subsequent hydrodemolition to remove the concrete material from inside the duct was apparently too aggressive

and shredded the plastic duct material within the girder web.

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

[x] Yes   [ ] No

If *Yes,* what design provisions are used to calculate the strength reduction?

[x] AASHTO LRFD Specifications   [x] AASHTO Segmental Bridge Specifications

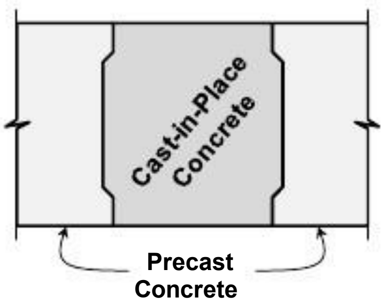
[ ] Other; please specify: one on one project; one, on another.

9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

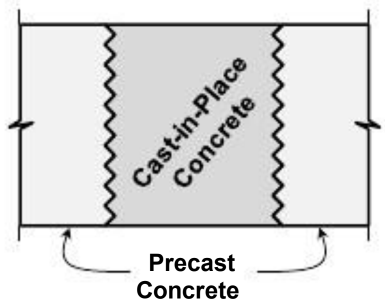
[ ] Yes   [x] No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

[x] Shear key: 100 %



[ ] Saw teeth: ________%



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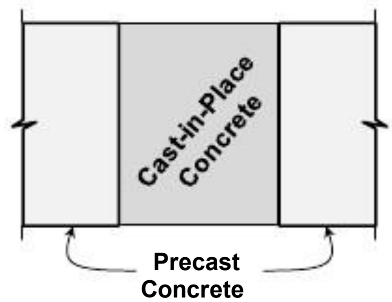

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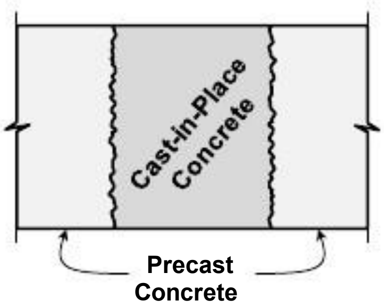
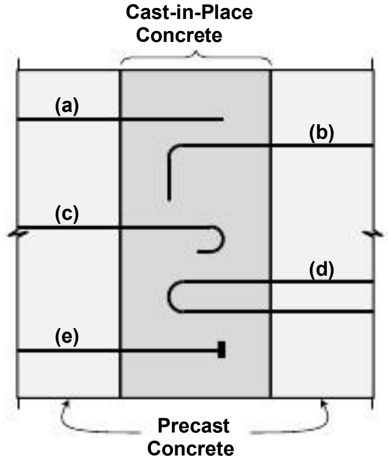


Please explain the factors that affect the type of interface that is chosen.

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

■ (a) Straight bars ☐ (b) 90-degree hooks ☐ (c) 180-degree hooks ■ (d) Hairpins

☐ (e) Headed bars ■ Other; please describe: Lapped embedded plates that were welded on one project.

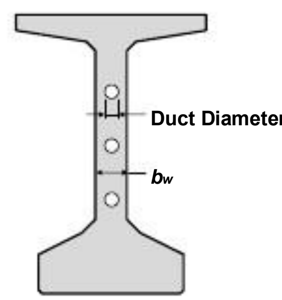


Please elaborate on the detailing of the interface reinforcement if necessary.

One project used hairpin and lapped embedded plates which were welded to each other to provide load transfer through the splice.

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
8"	3.25"	50 %
9"	3.7"	50 %
		%
		%
		%

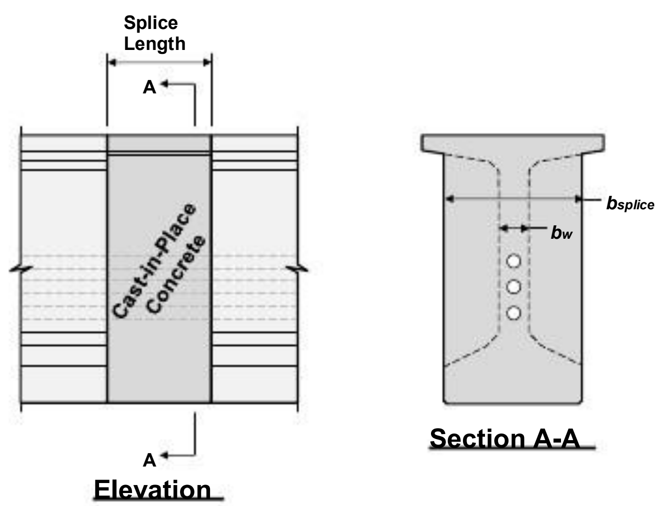


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

■ At the splice ☐ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	1'-0"	1'-0"	
$b_{splice} - b_w$			



Please explain the factors that affect the **length** of the splice region?

Want to minimize the length of the splice region to ease forming and casting but make it long enough to allow ducts to be spliced.

Access for coupling of post-tensioning ducts, proper consolidation and vibration of concrete and lapping/splicing of reinforcement.

Please explain the factors that affect the **width** of the splice region?

Diaphragms were cast at the splices

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ☒ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

■ Yes ☐ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

Some cracking in early splices, but it was traced to shoring that was allowing the pier segments to rotate slightly.

Misalignment of the lapping embedded plates.

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

Comments received from two separate projects from two consultants.

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here:

Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

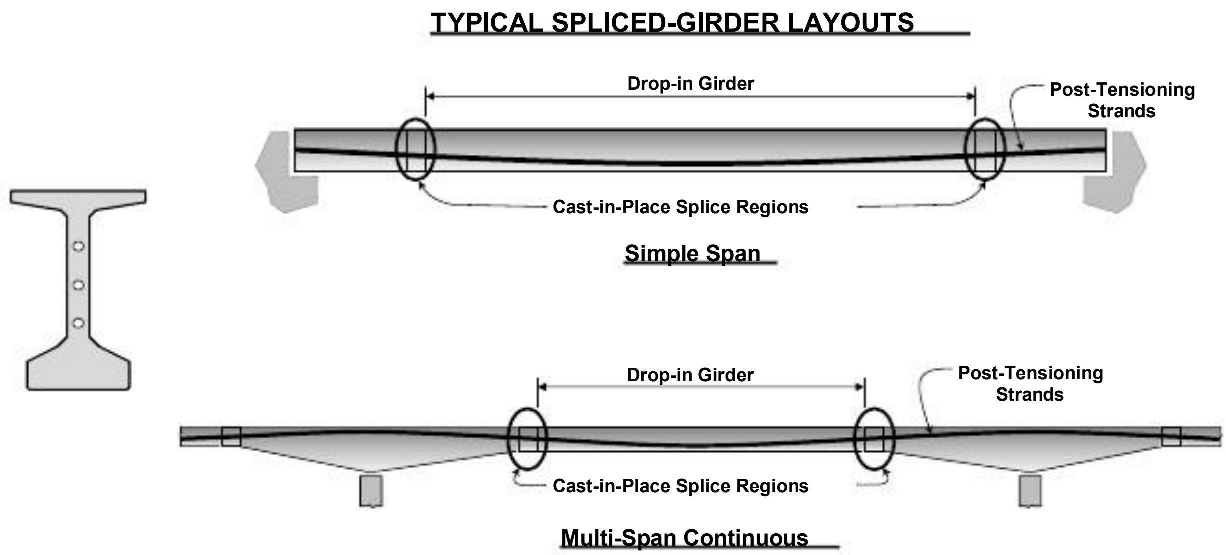
Ferguson Structural Engineering Laboratory at The University of Texas at Austin in Collaboration with the Texas Department of Transportation (TxDOT)

Survey of Spliced I-Girder Bridge Construction and Design Practices Focusing on Details of the Splice Region

The objective of the following survey is to identify design and detailing practices that have been successfully implemented within cast-in-place (CIP) splice regions of spliced I-girder bridges. Based on the best practices that are identified, a full-scale testing program will be conducted in an effort to develop splice region detailing standards for TxDOT.

Your response to the following survey will be invaluable to the research team. Please answer the questions as thoroughly as possible, providing details where necessary. The research results will be available in a final project report. Your time is greatly appreciated.

Please return this survey **by April 30** to: Greg Turco, TxDOT Bridge Division



Texas Department of Transportation Contact:	The University of Texas Research Team:
Greg Turco, PE **Address:** Bridge Division	Dr. Oguzhan Bayrak: bayrak@mail.utexas.edu Dr. James Jirsa: jirsa@uts.cc.utexas.edu Dr. Wassim Ghannoum: ghannoum@mail.utexas.edu Chris Williams: chrisw05@utexas.edu Andy Moore: ammoore@utexas.edu **Address:** Ferguson Structural Engineering Laboratory The University of Texas at Austin 10100 Burnet Rd., Building 177 Austin TX, 78758

Contact Information

Name of Person Completing the Survey: ______________________

Title: ______________________

State/District: Washington

Organization/Unit: DOT

Address: ______________________

____

Phone: ______________________

Fax: ______________________

Email: ______________________

A. General Information

1. Has your state/district had experience with the design and/or construction of spliced girder bridges?

   ☒ Yes ☐ No

   If *No,* has your state/district considered the use of spliced girder technology?

   ☐ Yes ☐ No

   If spliced girder technology is not **currently** being considered as a design option for new bridges, please explain why.

   ____

   ____

   ____

   ____

   ____

   ____

   ____

   If your state/district has had experience with spliced girder technology, please proceed with the remainder of the survey. If not, thank you for providing the above information.

2. How many spliced **I-girder** bridges have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☐ 11 to 20 ☒ Greater than 20

3. How many spliced **U-girder or box-girder** bridges (not including segmental bridges) have been constructed in your state?

   ☐ None ☐ 1 to 5 ☐ 6 to 10 ☒ 11 to 20 ☐ Greater than 20

4. Does your state have any spliced girder bridges with **curved U-girders or box girders?**

■ Yes ☐ No

5. Has your state used consulting engineers for the design of spliced girder bridges?

☐ Yes ■ No

If *Yes,* please list the consultant(s).

B. Design and Construction Practices of Spliced I-Girder Bridges

The following questions refer to the cast-in-place (CIP) splice regions located within the span lengths of spliced I-girder bridges.

6. For what percent of the spliced I-girder bridge projects in your state/district have the following duct materials been specified?

- Steel: 80 %
- Plastic (HDPE): 20 %

If one of the materials is preferred over the other, please explain why.

corrugated galvanized steel ducts are preferred because of dimensions fitting the web width in case of I-girders, and ease of placement compare to HDPE ducts.

7. Are you aware of any problems encountered **due to the duct material** that was chosen for a particular project?

☐ Yes ■ No

If *Yes,* please briefly describe the problem(s).

8. Does your state/district consider a reduction in shear strength due to the presence of the post-tensioning duct in the girder web?

[x] Yes [ ] No

If *Yes,* what design provisions are used to calculate the strength reduction?

[x] AASHTO LRFD Specifications [ ] AASHTO Segmental Bridge Specifications

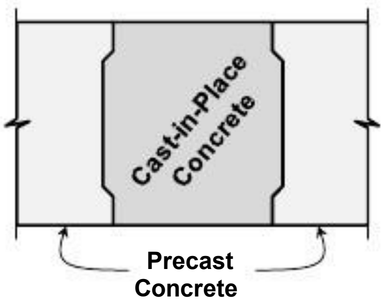
[ ] Other; please specify: ______________________________

9. Has your state/district ever used ungrouted ducts in spliced I-girder construction?

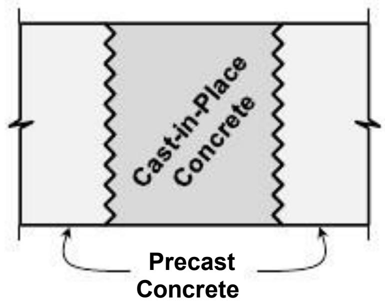
[ ] Yes [x] No

10. What shear transfer mechanism has been used in your state/district at the interface between the pretensioned I-girders and the cast-in-place splice region? Select all that apply, and estimate the percent of projects for which each interface has been specified.

[ ] Shear key: ________%



[x] Saw teeth: 100 %



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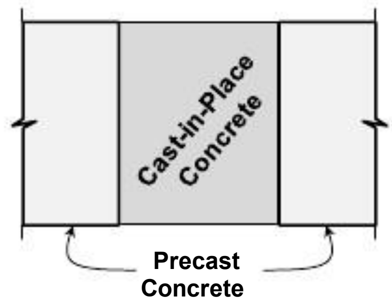
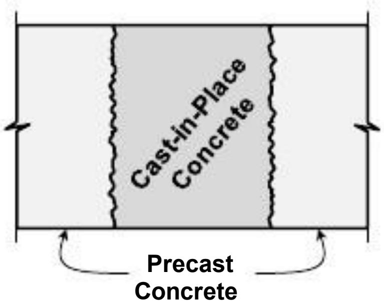


[ ] Sandblasting or intentional roughening: ________%

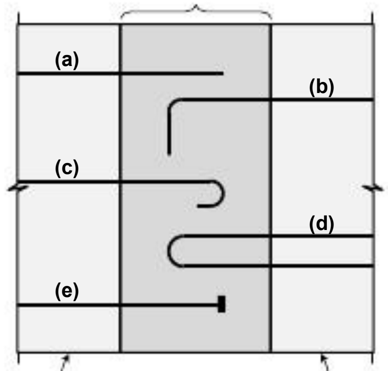


Please explain the factors that affect the type of interface that is chosen.

The saw teeth shear key is commonly used

11. How is the longitudinal reinforcement crossing the splice interface of I-girders **typically** detailed (refer to the figure below)? More than one answer can be selected.

- [x] (a) Straight bars
- [ ] (b) 90-degree hooks
- [ ] (c) 180-degree hooks
- [ ] (d) Hairpins
- [ ] (e) Headed bars
- [ ] Other; please describe: ______

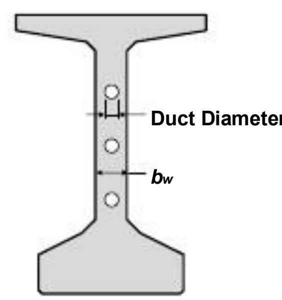


Please elaborate on the detailing of the interface reinforcement if necessary.

The closure is wide enough to allow lap splice.

12. Please provide the combinations of the girder web width, b_w, and nominal duct diameter that have been used for spliced I-girder construction in your state/district. Estimate the percent of projects for which each combination has been specified.

Web Width, b_w	Duct Diameter	Percent of Projects
8.0 in. (I-Girder)	4.25 in.	50 %
10.0 in (Tubs)	4.25	50 %
		%
		%
		%

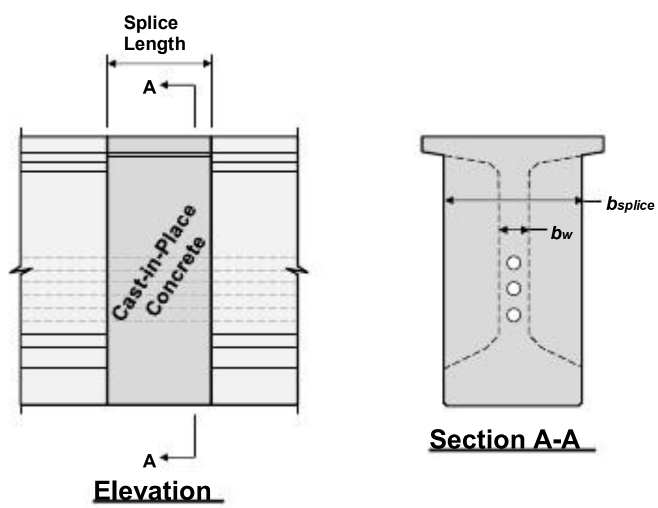


13. Where does your state/district prefer to locate transverse diaphragms (refer to the figure below)?

☐ At the splice ☒ Away from the splice ☐ No preference

14. Please provide the minimum, maximum, and typical values specified for the length and width of the CIP splice region (refer to the figure below). For the width of the splice region (i.e., $b_{splice} - b_w$), **consider only cases when a transverse diaphragm is not located at the splice.** If transverse diaphragms are always located at the splice region, the ($b_{splice} - b_w$) cells of the table can be left blank.

Splice Region	Minimum	Maximum	Typical
Length	2.0 ft	Special cases	2.0 ft
$b_{splice} - b_w$	8 " (I-Girders), 10" (Tubs)	Special cases	8 " (I-Girders), 10" (Tubs)



Please explain the factors that affect the **length** of the splice region?

Suitability for duct splicing, bar splicing and casting concrete.

Please explain the factors that affect the **width** of the splice region?

Adjacent precast girders width

15. Have you had any serviceability/aesthetic issues (e.g., cracking, discolored concrete, etc.) related to the splice region?

☐ Yes ■ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

16. Have you had any constructability issues (e.g., concrete consolidation, formwork, shoring, etc.) related to the splice region?

☒ Yes ☐ No

If *Yes,* please briefly describe the issue(s) and any actions that have been taken to resolve the problem(s).

Concrete consolidation in one project. Achieving high strength concrete at splice.

17. Please provide any additional information regarding your experience with the implementation of spliced girder technology that you believe may be useful to the research team. Feel free to give specific examples regarding any aspect of the design and construction of spliced girder bridges.

5.9.2 WSDOT Criteria for Use of Spliced Girders

http://www.wsdot.wa.gov/publications/manuals/fulltext/M23-50/BDM.pdf

C. Request for Additional Material

If possible, please attach **drawings of existing spliced girder bridges** in your state.

If your state/district has specific **design guidelines/requirements for spliced girder bridges,** please submit this material with the survey. Alternatively, a web link to the guidelines/requirements can be provided here: http://www.wsdot.wa.gov/publications/manuals/fulltext/M23-50/BDM.pdf

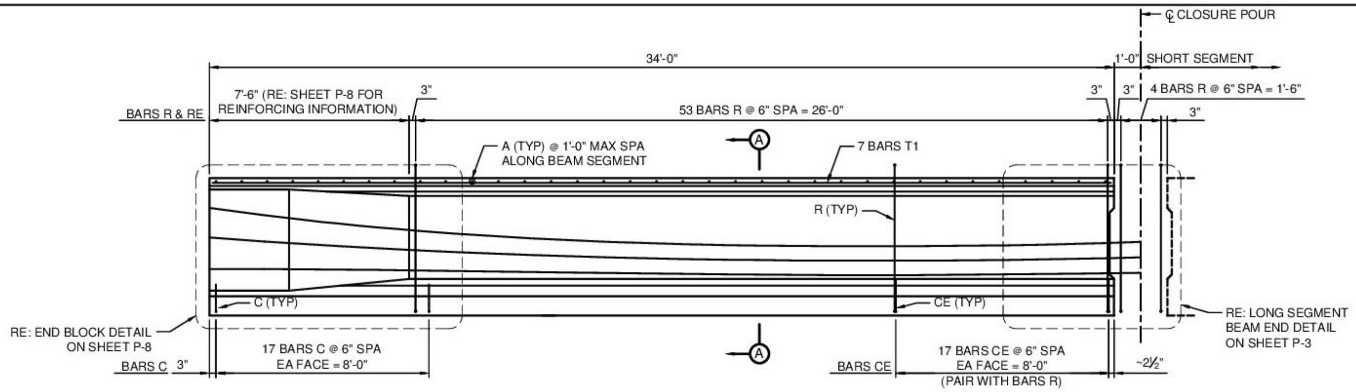
Any other relevant information you can offer the research team will be greatly appreciated.

Please upload supplemental material to https://ftp.dot.state.tx.us/dropbox/.

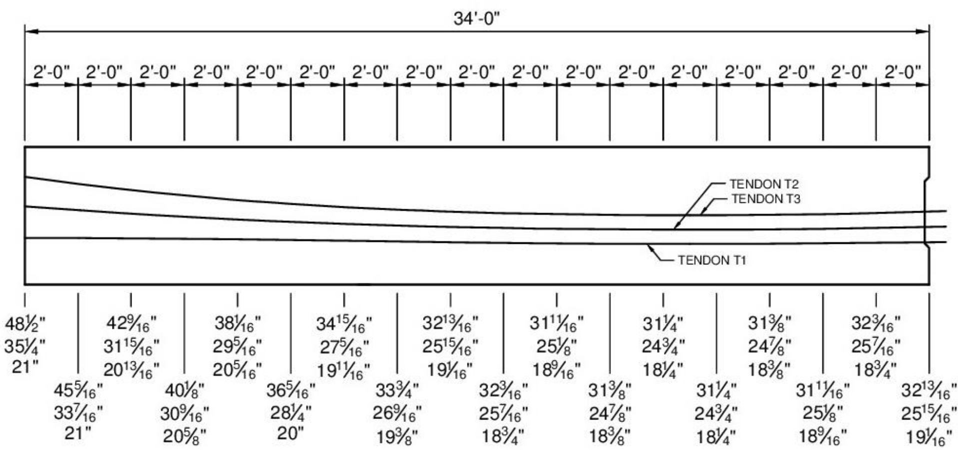
Appendix B. Test Specimen Drawings

INTRODUCTION

Detailed drawings of the spliced girder test specimens of the cast-in-place (CIP) splice region experimental program are provided in this appendix. The cross-section of the specimens outside the end blocks was based on the geometry of a Tx62 girder except that all horizontal (i.e., transverse) dimensions were increased by 2 in. Current Tx62 details can be accessed online from TxDOT (*Texas Department of Transportation Bridge Division: Prestressed Concrete I-Girder Details*).



ELEVATION - LONG SEGMENT



TENDON PROFILES - LONG SEGMENT

NOTES:

1. TENDON PROFILE DIMENSIONS ARE FROM BOTTOM OF GIRDER TO CENTERLINE OF DUCTS

Ferguson Structural Engineering Laboratory

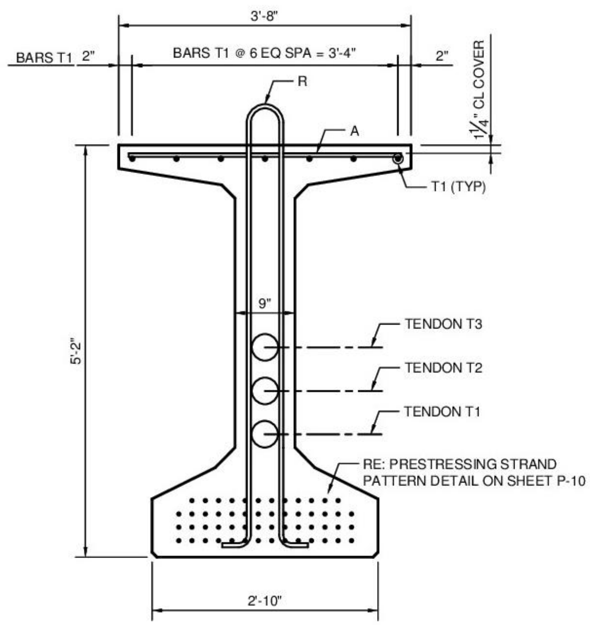
SPLICE REGION TEST SPECIMENS

TxDOT SPLICED GIRDER PROJECT

Project #: 0-6652

ELEVATIONS - LONG SEGMENT

P-1



SECTION A-A

Ferguson Structural Engineering Laboratory

SPLICE REGION TEST SPECIMENS

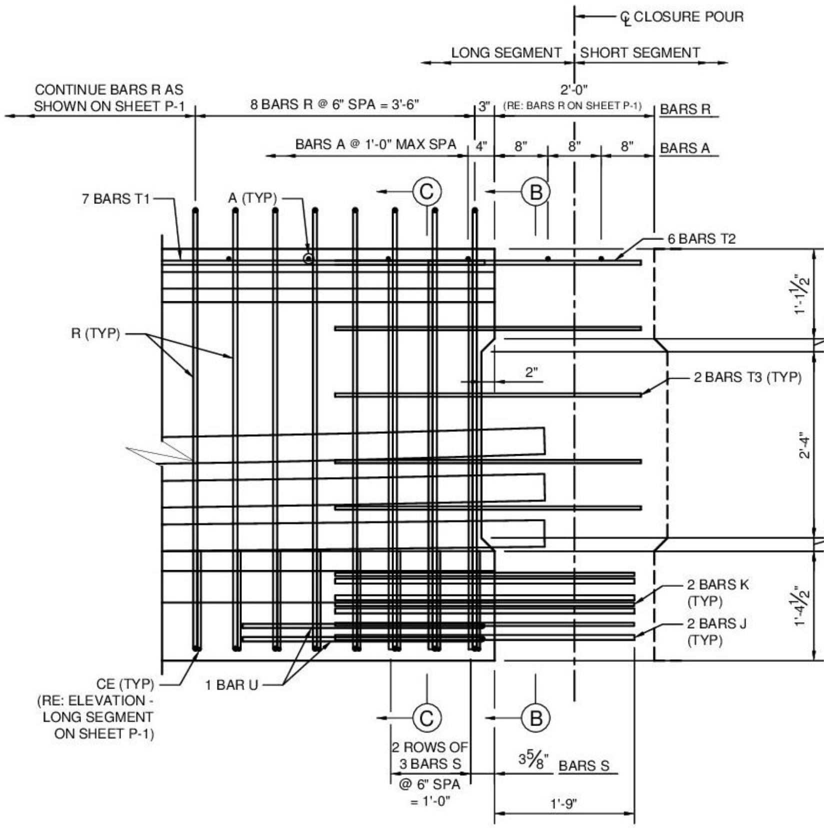
TxDOT SPLICED GIRDER PROJECT

Project #: 0-6652

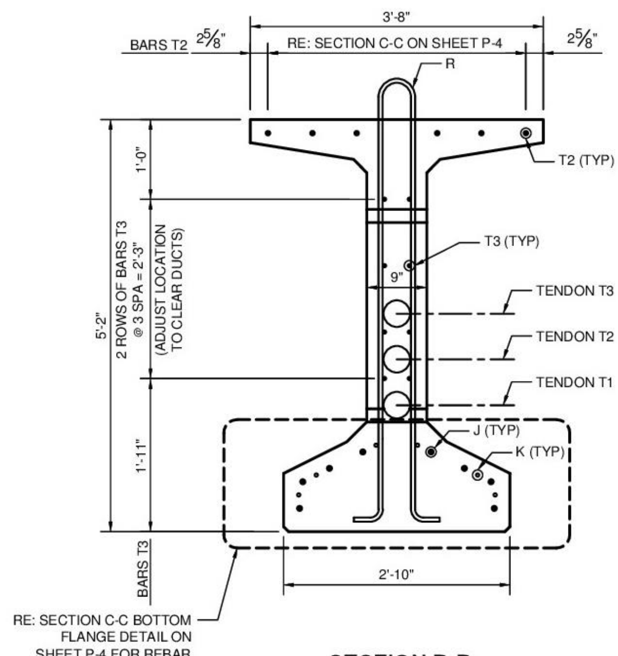
SECTION A-A

P-2

336



LONG SEGMENT BEAM END DETAIL



SECTION B-B

(SHOWING MILD REINFORCEMENT EXTERIOR TO PRECAST SEGMENTS)

NOTES:

1. EXTEND DUCTS 8" FROM BEAM FACE
2. CUT PRETENSIONED STRANDS WITHIN 3" OF BEAM FACE

Ferguson Structural Engineering Laboratory

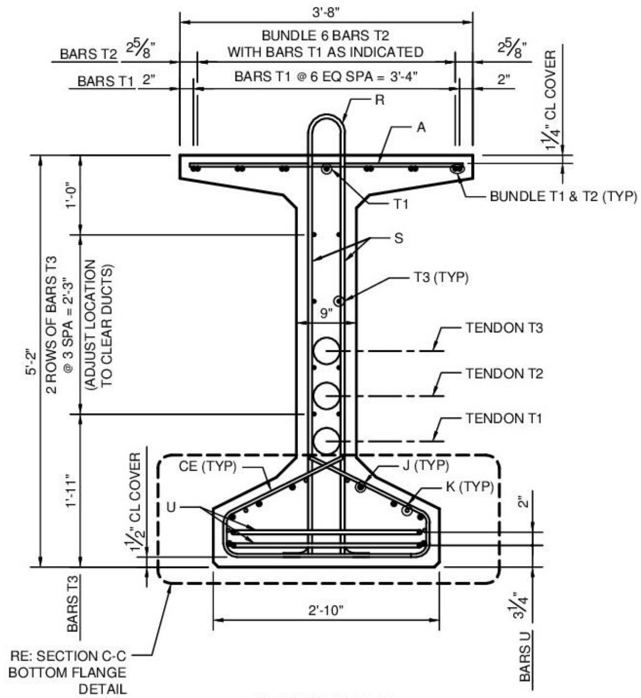
SPLICE REGION TEST SPECIMENS

TxDOT SPLICED GIRDER PROJECT

Project #: 0-6652

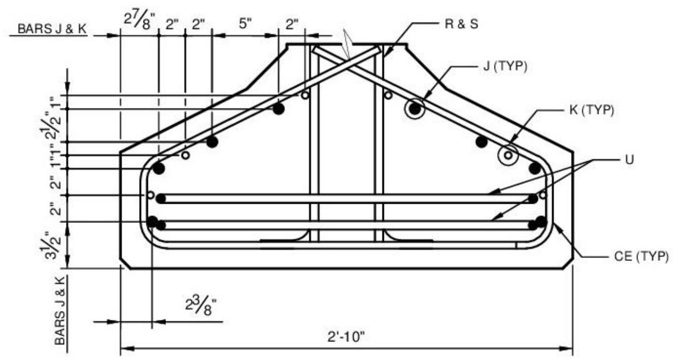
LONG SEGMENT BEAM END DETAILS

P-3



SECTION C-C

(PRESTRESSING NOT SHOWN FOR CLARITY)



SECTION C-C BOTTOM FLANGE DETAIL

(PRESTRESSING NOT SHOWN FOR CLARITY)

Ferguson Structural Engineering Laboratory

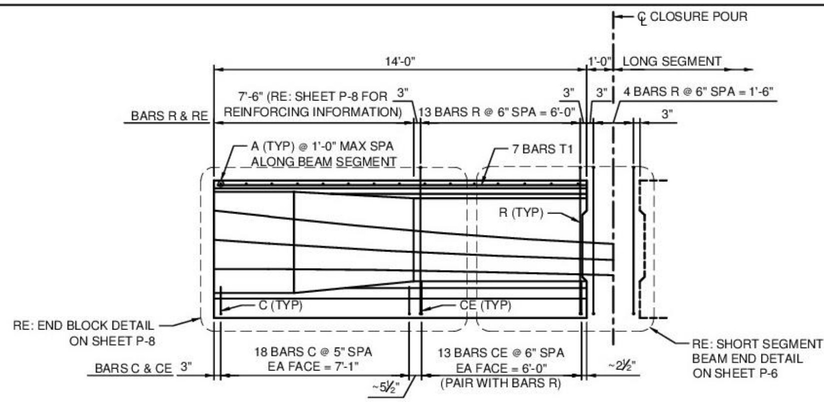
SPLICE REGION TEST SPECIMENS

TxDOT SPLICED GIRDER PROJECT

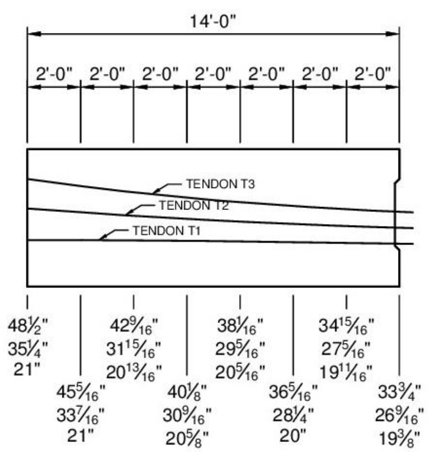
Project #: 0-6652

SECTION C-C

P-4



ELEVATION - SHORT SEGMENT



TENDON PROFILES - SHORT SEGMENT

NOTES:

1. TENDON PROFILE DIMENSIONS ARE FROM BOTTOM OF GIRDER TO CENTERLINE OF DUCTS

Ferguson Structural Engineering Laboratory

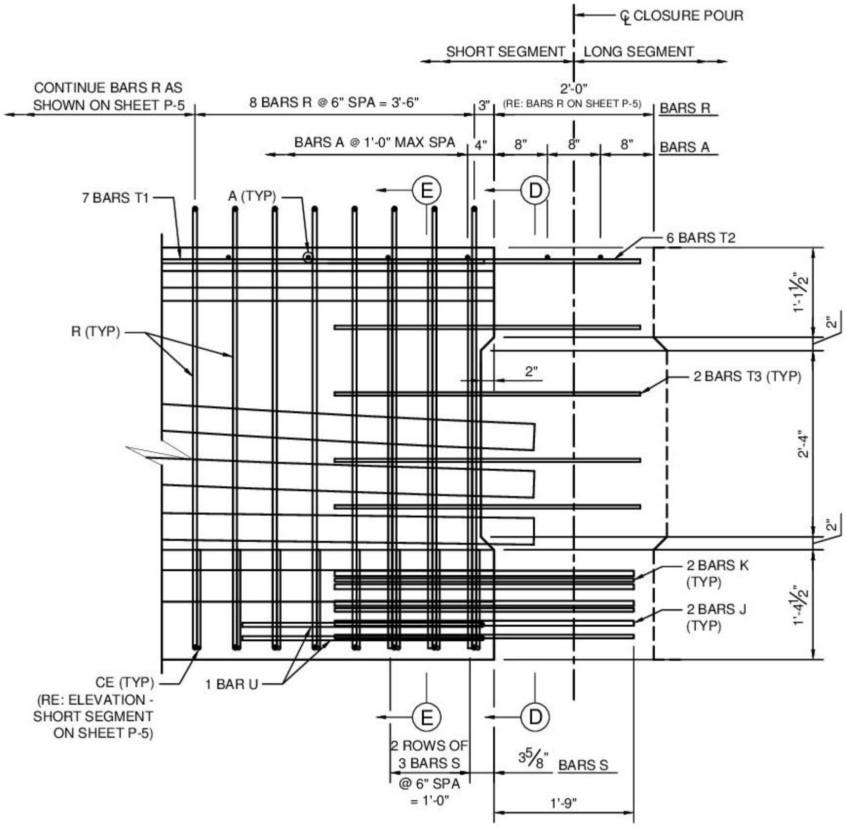
SPLICE REGION TEST SPECIMENS

TxDOT SPLICED GIRDER PROJECT

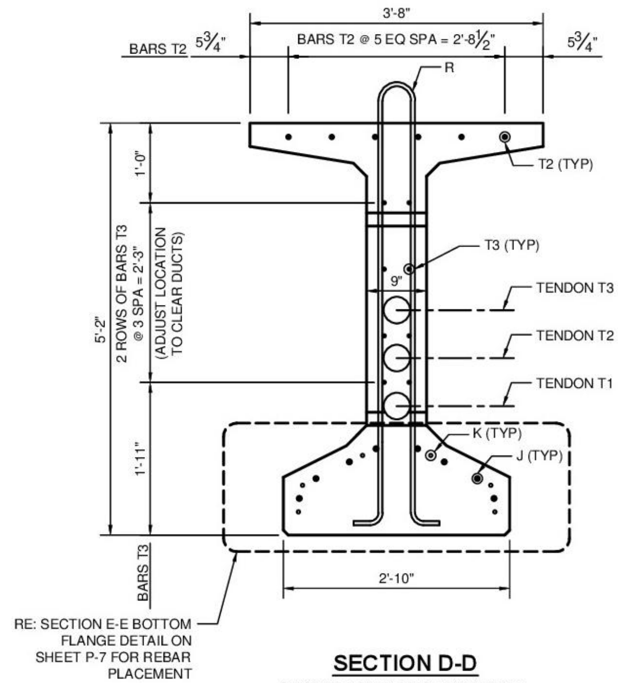
Project #: 0-6652

ELEVATIONS - SHORT SEGMENT

P-5



SHORT SEGMENT BEAM END DETAIL



SECTION D-D

(SHOWING MILD REINFORCEMENT EXTERIOR TO PRECAST SEGMENTS)

NOTES:

1. EXTEND DUCTS 8" FROM BEAM FACE
2. CUT PRETENSIONED STRANDS WITHIN 3" OF BEAM FACE

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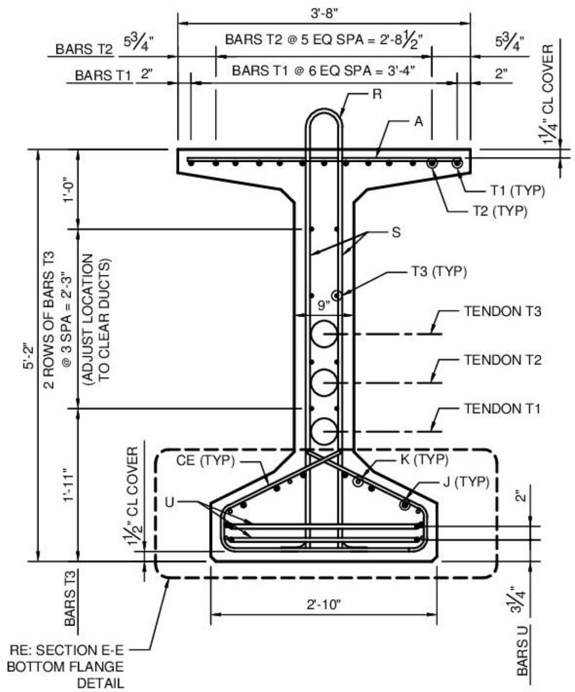
SPLICE REGION TEST SPECIMENS

TxDOT SPLICED GIRDER PROJECT

Project #: 0-6652

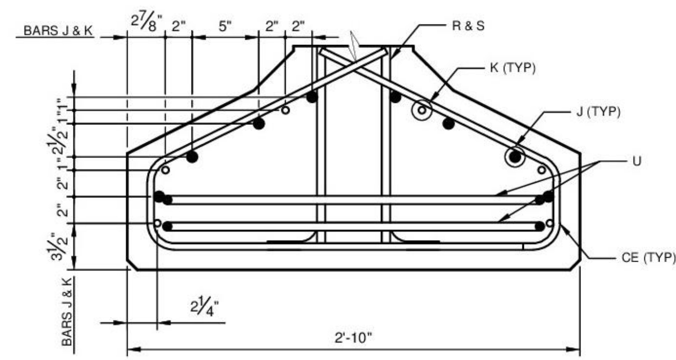
SHORT SEGMENT BEAM END DETAILS

P-6



SECTION E-E

(PRESTRESSING NOT SHOWN FOR CLARITY)



SECTION E-E BOTTOM FLANGE DETAIL

(PRESTRESSING NOT SHOWN FOR CLARITY)

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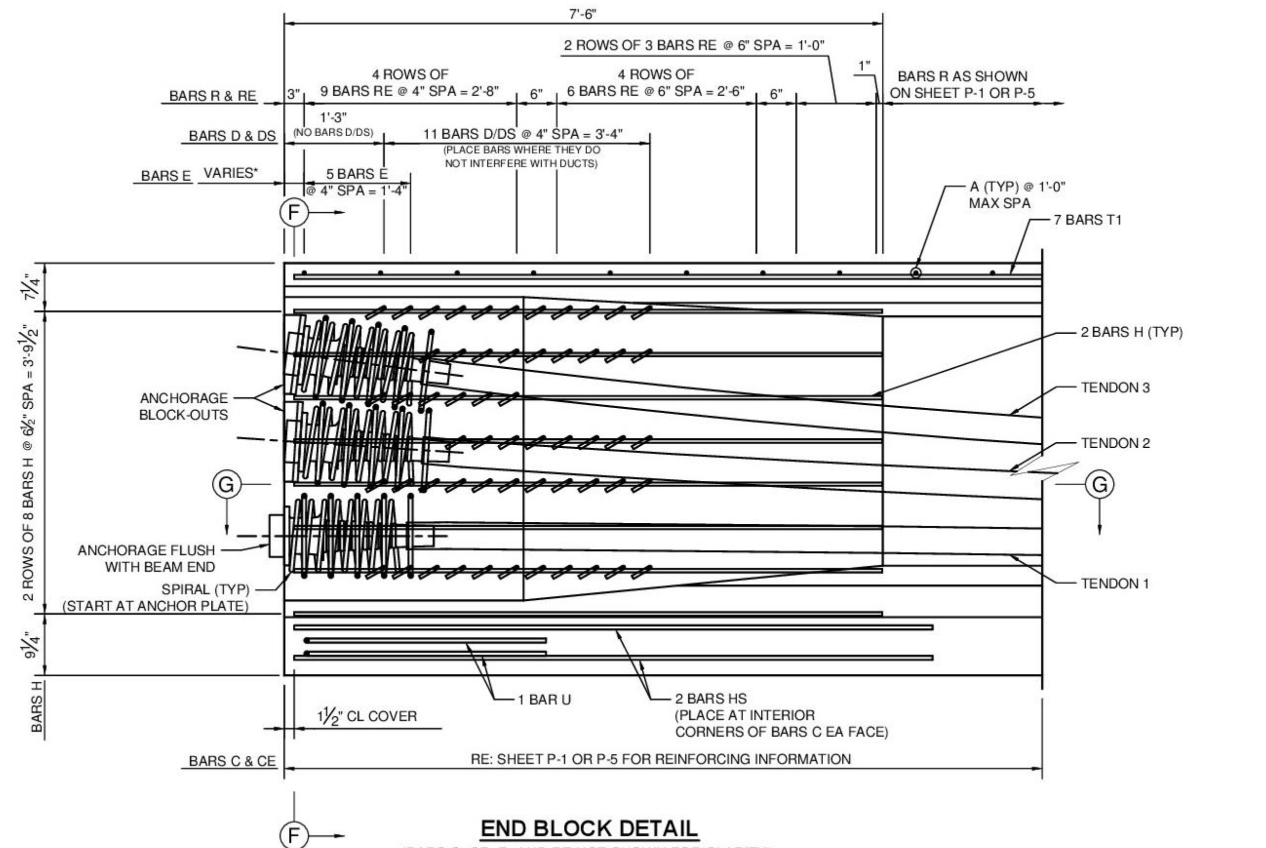
SPLICE REGION TEST SPECIMENS

TxDOT SPLICED GIRDER PROJECT

Project #: 0-6652

SECTION E-E

P-7



END BLOCK DETAIL

(BARS C, CE, R, AND RE NOT SHOWN FOR CLARITY)

NOTES:

* BEGIN BARS E AT 3" FROM OUTSIDE SURFACE OF ANCHOR PLATE

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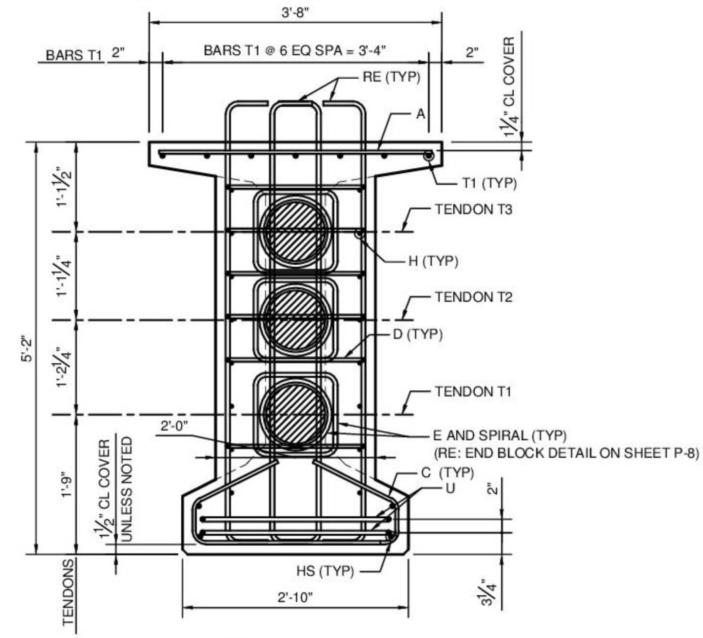
SPLICE REGION TEST SPECIMENS

TxDOT SPLICED GIRDER PROJECT

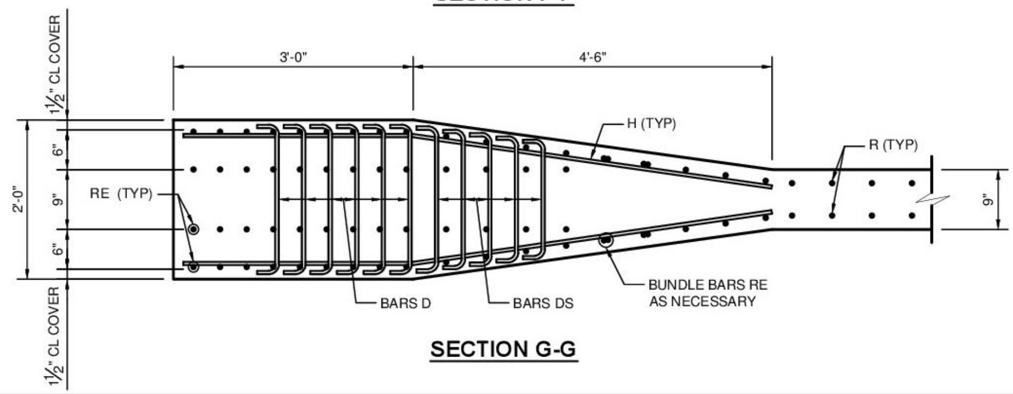
Project #: 0-6652

END BLOCK ELEVATION

P-8



SECTION F-F



SECTION G-G

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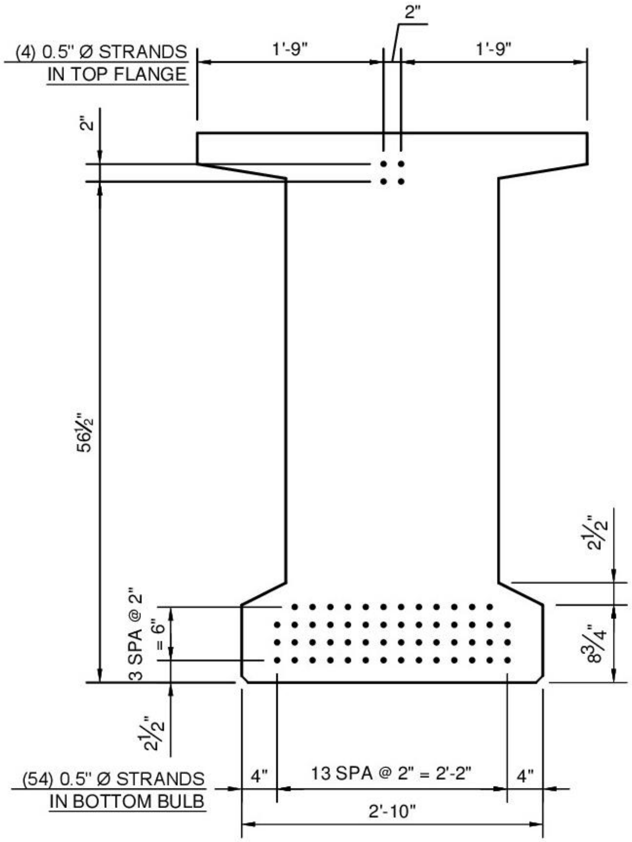
SPLICE REGION TEST SPECIMENS

TxDOT SPLICED GIRDER PROJECT

Project #: 0-6652

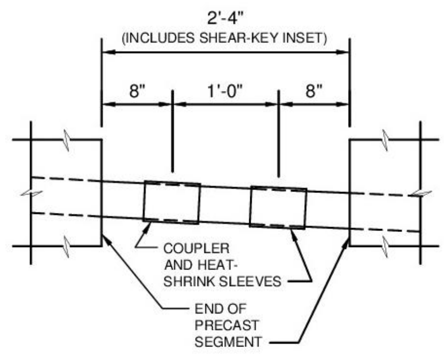
SECTIONS F-F AND G-G (END BLOCK)

P-9



PRESTRESSING STRAND PATTERN DETAIL
(58-0.5" Ø STRANDS)

• FULLY BONDED STRAND



DUCT SPLICE DETAIL

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SPLICE REGION TEST SPECIMENS
TxDOT SPLICED GIRDER PROJECT

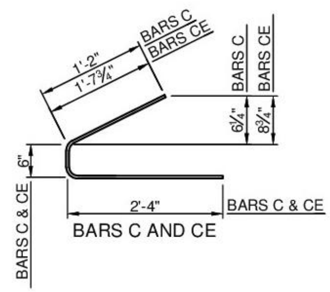
Project #: 0-6652

STRAND PATTERN DETAIL

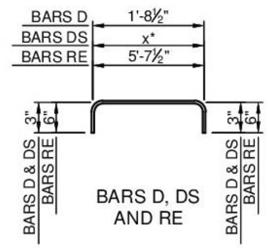
P-10

REINFORCING STEEL			
BAR	SIZE	NO. OF BARS (SHORT SEGMENT)	NO. OF BARS (LONG SEGMENT)
A	3	15	39†
C	4	36	34
CE	4	26	34
D	4	36	36
DS	4	30	30
E	5	15	15
H	4	16	16
HS	5	4	4
J	6	8	8
K	4	6	6
R	5	18†	62†
RE	5	66	66
S	5	11†	11†
T1	4	7	14***
T2	4 OR 5††	6	6
T3	4	8	8
U	5	4	4
SPIRAL	5	3	3

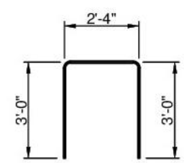
* DIMENSION "x" OF FIVE DS BARS: 1'-8", 1'-7", 1'-6", 1'-5", AND 1'-4'
** LAP BARS WHERE NECESSARY
*** MAY VARY DEPENDING ON LENGTH OF INDIVIDUAL BARS
† EXTRA BARS ARE NEEDED IN ORDER TO BE TAKEN TO THE UNIVERSITY OF TEXAS
†† BARS T2 ARE NO. 4 FOR GIRDER 1 AND NO. 5 FOR GIRDER 2



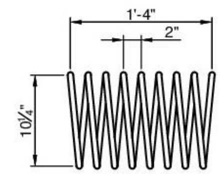
BARS C AND CE



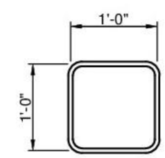
BARS D, DS AND RE



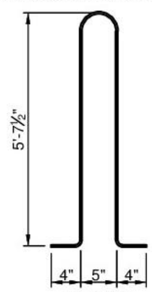
BARS U



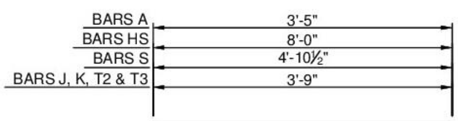
SPIRAL



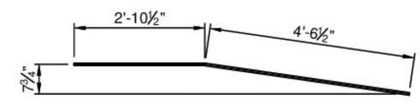
BARS E



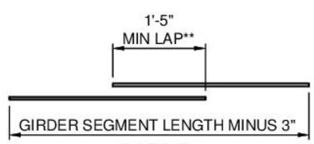
BARS R



BARS A, HS, J, K, S, T2 AND T3



BARS H



BARS T1

GENERAL NOTES

PRE-TENSIONED STEEL:
1. ALL PRE-TENSIONED STRANDS SHALL BE 0.5" DIA. ASTM A416 LOW RELAXATION STRANDS, f_{pu} = 270 KSI
2. ALL STRANDS SHALL BE BROUGHT TO FULL TENSION ($0.75f_{pu}$)

POST-TENSIONED TENDONS:
1. 4" PLASTIC DUCT SHALL BE USED
2. ANCHORAGES, DUCTS, COUPLERS, AND SPIRAL REINFORCEMENT WILL BE PROVIDED TO THE PRECASTER BY THE UNIVERSITY OF TEXAS

REINFORCING STEEL:
1. ALL MILD REINFORCING STEEL SHALL BE ASTM A615 GR 60, f_y = 60 KSI
2. ONLY DEFORMED BARS SHALL BE USED, NO WELDED WIRE REINFORCEMENT

CONCRETE:
1. CONCRETE FOR PRECAST GIRDER SEGMENTS: f'_c = 12.0 KSI (AT 28 DAYS), f'_{ci} = 6.0 KSI

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SPLICE REGION TEST SPECIMENS

TxDOT SPLICED GIRDER PROJECT

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REINFORCING DETAILS AND GENERAL NOTES

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Appendix C. Spliced Girder Specimen Shear Strength Calculations

INTRODUCTION AND NOTATION

A summary of the shear strength calculations for the spliced girder test specimens of the cast-in-place (CIP) splice region experimental program are provided in the table below. All values correspond with the critical section located at the splice region interface. The variables used in the tables are defined as follows (adopted from AASHTO LRFD (2014)):

A_{ps} = Area of prestressing steel on the flexural tension side of the member (in.^{2})

A_v = Area of shear reinforcement within distance s (in.^{2})

b_v = Effective web width equal to the minimum web width within depth d_v and adjusted for the presence of post-tensioning ducts (in.)

b_w = Gross web width (in.)

d_e = Effective depth measured from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement (in.)

d_v = Effective shear depth equal to the distance measured perpendicular to the neutral axis between the resultants of the tensile and compressive forces due to flexure, not to be taken as less than the greater of $0.9d_e$ or $0.72h$ (in.)

f'_c = Compressive strength of concrete at the time of testing (ksi)

f_{po} = Average of the stress in each post-tensioning tendon of the test girders after the anchorage is set (see Section 4.14.1) (ksi)

f_y = Measured yield strength of transverse reinforcement (ksi)

h = Overall member depth (in.)

M_{sw} = Moment at the critical section due to the self-weight of the girder (kip-in.)

M_u = Factored moment at the critical section, not taken as less than $(V_u – V_p)d_v$ (kip-in.)

s = Spacing of transverse reinforcement measured in the direction parallel to the longitudinal reinforcement (in.)

V_c = Nominal shear resistance provided by tensile stresses in the concrete (kip)

V_n = Nominal shear resistance at the critical section (kip)

V_p = Vertical component of the post-tensioning force (kip)

V_s = Nominal shear resistance provided by shear reinforcement (kip)

V_{sw} = Shear at the critical section due to self-weight of the girder (kip)

V_{test} = Maximum shear force at the critical section during testing (kip)

V_u = Factored shear force at the critical section (kip)

β = Factor indicating ability of diagonally cracked concrete to transmit tension and shear

ε_s = Net longitudinal tensile strain at the centroid of the tension reinforcement (in./in.)

θ = Angle of inclination of diagonal compressive stresses (degrees)

λ_{duct} = Proposed shear strength reduction factor to account for the effect of the presence of a post-tensioning duct in the member's web

$Ø_{duct}$ = Diameter of post-tensioning duct (in.)

Shear Strength Calculations Using AASHTO LRFD (2014) General Shear Procedure

Test Speci-men	f'_c (ksi)	b_w (in.)	$Ø_{duct}$ (in.)	b_v (in.)	d_v* (in.)	M_{sw} (kip-in.)	V_{sw} (kip)	M_u (kip-in.)	V_u (kip)	A_{ps} (in.^{2})	f_{po} (ksi)	ε_s (in./in.) $\times 10^3$	β	V_c (kip)	$A_v f_y$ (kip)	s (in.)	θ (deg.)	V_s (kip)	V_p (kip)	Max. V_n (kip)	V_n (kip)	$\frac{V_{test}}{V_n}$
No. 1	9.48	9	4	8	50.4	2027	20.5	58479	574	7.812	186.8	1.088	2.64	104	38.4	6	32.8	501	33.1	989	638	1.04
No. 2	10.07	9	4	8	50.4	2037	20.6	60167	591	7.812	185.3	1.364	2.37	95.9	42.0	6	33.8	527	33.0	1048	656	1.07

* d_v is governed by the expression $0.72h$

Shear Strength Calculations Using Proposed Modifications to AASHTO LRFD (2014) General Shear Procedure

Test Spec.	f'_c (ksi)	b_w (in.)	$Ø_{duct}$ (in.)	d_v* (in.)	M_{sw} (kip-in.)	V_{sw} (kip)	M_u (kip-in.)	V_u (kip)	A_{ps} (in.^{2})	f_{po} (ksi)	ε_s (in./in.) $\times 10^3$	β	V_c (kip)	$A_v f_y$ (kip)	s (in.)	θ (deg.)	λ_{duct}	$\lambda_{duct} V_s$ (kip)	V_p (kip)	Max. V_n (kip)	V_n (kip)	$\frac{V_{test}}{V_n}$
No. 1	9.48	9	4	50.4	2027	20.5	51654	507	7.812	186.8	0.1788	4.23	187	38.4	6	29.6	0.60	343	33.1	1108	563	1.18
No. 2	10.07	9	4	50.4	2037	20.6	52515	515	7.812	185.3	0.3453	3.81	173	42.0	6	30.2	0.60	366	33.0	1175	573	1.23

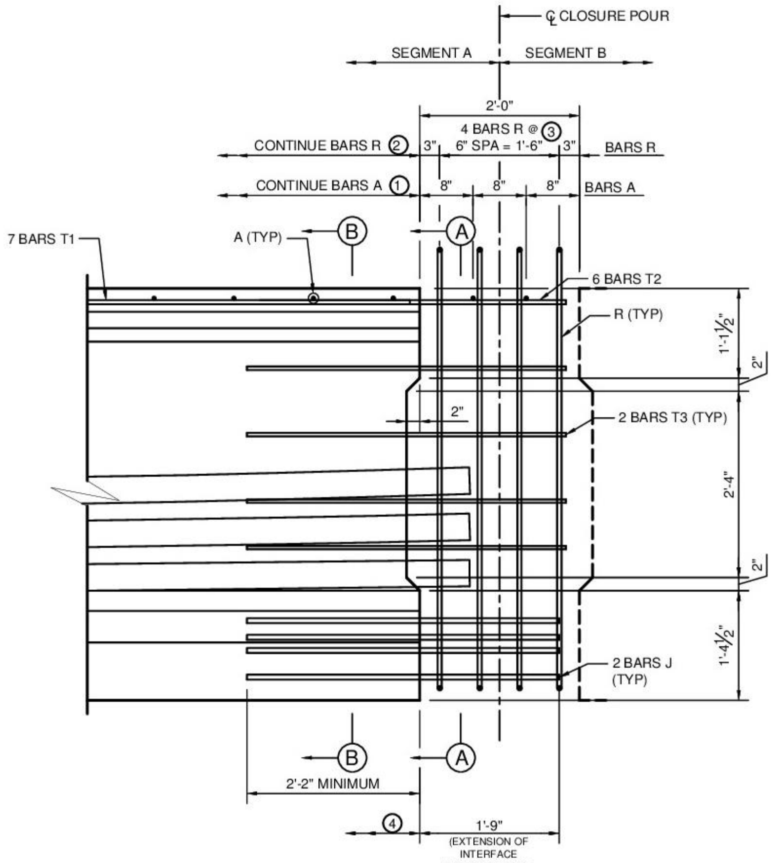
* d_v is governed by the expression $0.72h$

Calculation of the nominal shear resistance, V_n, using the AASHTO LRFD (2014) general shear procedure and the proposed provisions is dependent on the shear force and corresponding moment, V_u and M_u, acting at the critical section. To be consistent with the nature of the specifications, the shear V_u is equal to ϕV_n in the above calculations, where ϕ is the resistance factor for shear (0.90). M_u is the moment that acts simultaneously with V_u. Calculation of V_n therefore requires an iterative approach. The same methodology has been followed by past shear researchers (Hovell, 2011; Moore, 2014; Nakamura, Avendaño, and Bayrak, 2013).

Appendix D. Example Splice Region Details

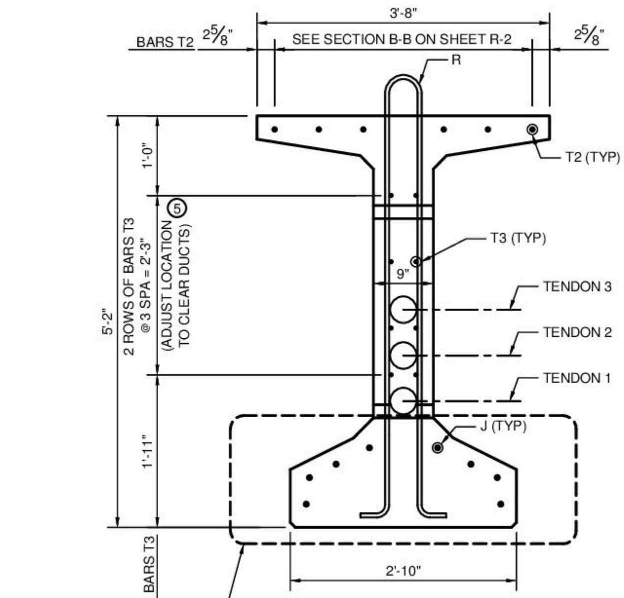
INTRODUCTION

Detailed drawings of the cast-in-place (CIP) splice region details developed as a result of the spliced girder research program are presented in this appendix. The details correspond with the recommendations discussed in Chapter 7. The cross-section of the girder shown in the example details is based on the geometry of a Tx62 girder except that all horizontal (i.e., transverse) dimensions have been increased by 2 in. to accommodate the post-tensioning ducts in the web. Current Tx62 details can be accessed online from TxDOT (*Texas Department of Transportation Bridge Division: Prestressed Concrete I-Girder Details*).



SEGMENT A BEAM END DETAIL

(BARS CE, S, AND U OF END REGION REINFORCEMENT NOT SHOWN)



SECTION A-A

(SHOWING MILD REINFORCEMENT EXTERIOR TO PRECAST SEGMENTS)

GENERAL NOTES:

1. CUT PRETENSIONED STRANDS 2" TO 3" FROM BEAM FACE
2. TENDON LOCATIONS MAY VARY FROM THOSE SHOWN

1 CONTINUE BARS A IN ACCORDANCE WITH TxDOT STANDARDS

2 CONTINUE BARS R IN ACCORDANCE WITH TxDOT STANDARDS/DESIGN REQUIREMENTS

3 SPACING OF BARS R IN SPLICE REGION MAY VARY (SEE GENERAL NOTES ON SHEET R-6)

4 PROVIDE BARS CE, S, AND U IN ACCORDANCE WITH TxDOT STANDARDS/DESIGN REQUIREMENTS

5 PLACEMENT OF BARS T3 SHOULD MATCH THAT OF ADJACENT GIRDER SEGMENT

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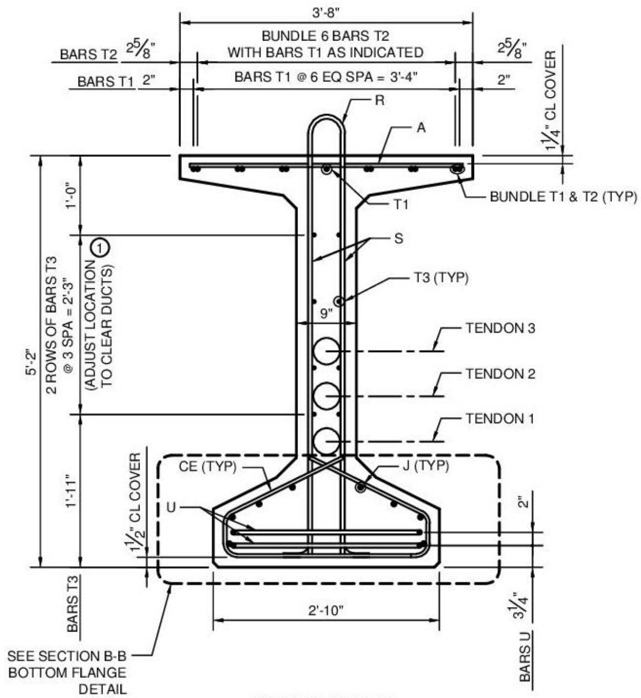
SPLICE REGION DETAILS

TxDOT SPLICED GIRDER PROJECT

Project #: 0-6652

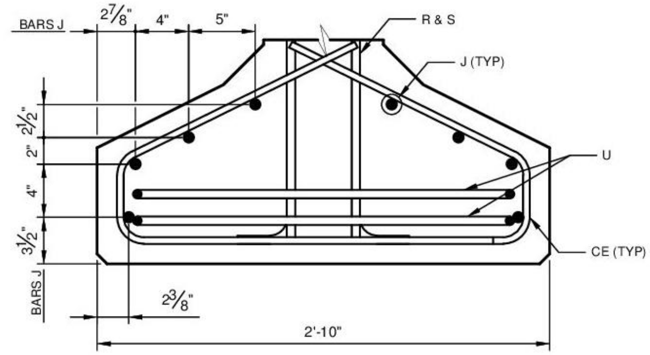
SEGMENT A BEAM END DETAILS

R-1



SECTION B-B

(PRESTRESSING NOT SHOWN)



SECTION B-B BOTTOM FLANGE DETAIL

(PRESTRESSING NOT SHOWN)

GENERAL NOTES:

1. TENDON LOCATIONS MAY VARY FROM THOSE SHOWN

1 PLACEMENT OF BARS T3 SHOULD MATCH THAT OF ADJACENT GIRDER SEGMENT

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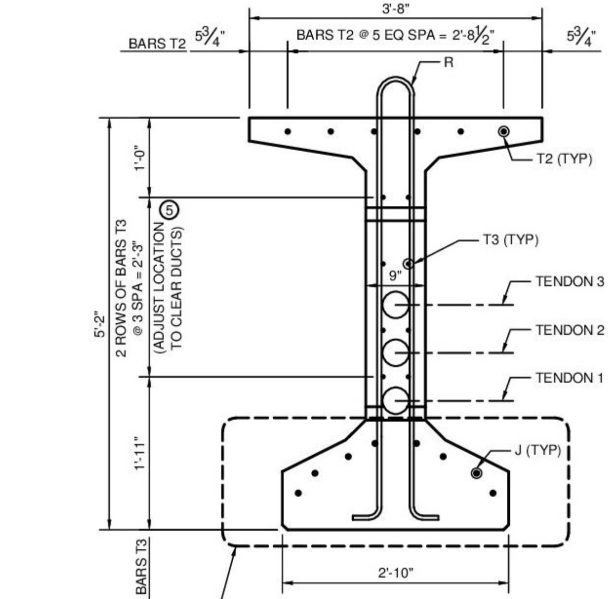
SPLICE REGION DETAILS

TxDOT SPLICED GIRDER PROJECT

Project #: 0-6652

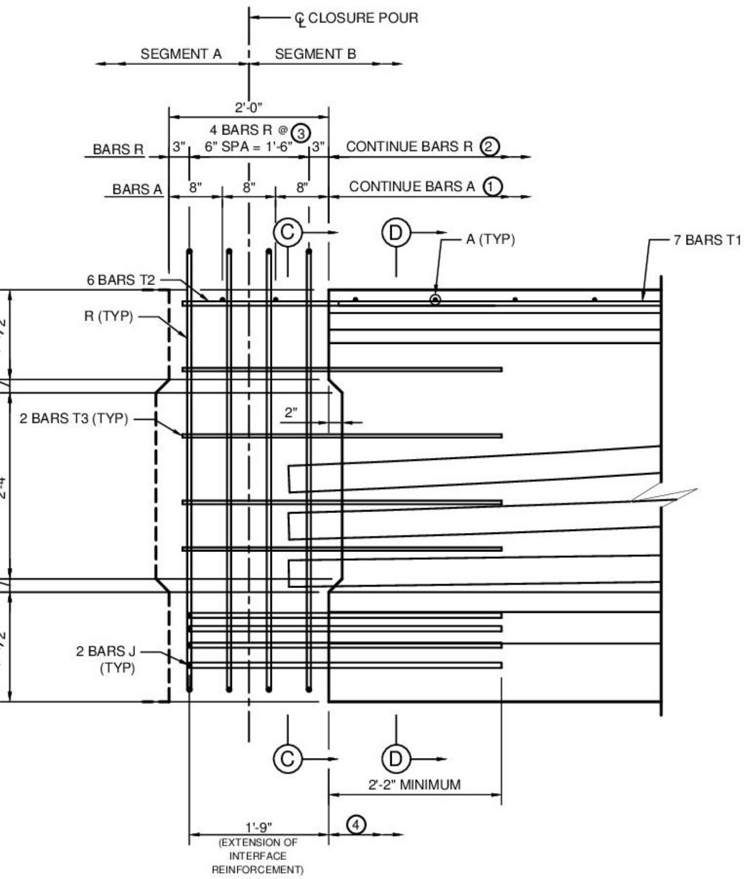
SECTION B-B

R-2



SECTION C-C

(SHOWING MILD REINFORCEMENT EXTERIOR TO PRECAST SEGMENTS)



SEGMENT B BEAM END DETAIL

(BARS CE, S, AND U OF END REGION REINFORCEMENT NOT SHOWN)

1 CONTINUE BARS A IN ACCORDANCE WITH TxDOT STANDARDS

2 CONTINUE BARS R IN ACCORDANCE WITH TxDOT STANDARDS/DESIGN REQUIREMENTS

3 SPACING OF BARS R IN SPLICE REGION MAY VARY (SEE GENERAL NOTES ON SHEET R-6)

4 PROVIDE BARS CE, S, AND U IN ACCORDANCE WITH TxDOT STANDARDS/DESIGN REQUIREMENTS

5 PLACEMENT OF BARS T3 SHOULD MATCH THAT OF ADJACENT GIRDER SEGMENT

GENERAL NOTES:

1. CUT PRETENSIONED STRANDS 2" TO 3" FROM BEAM FACE
2. TENDON LOCATIONS MAY VARY FROM THOSE SHOWN

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SPLICE REGION DETAILS

TxDOT SPLICED GIRDER PROJECT

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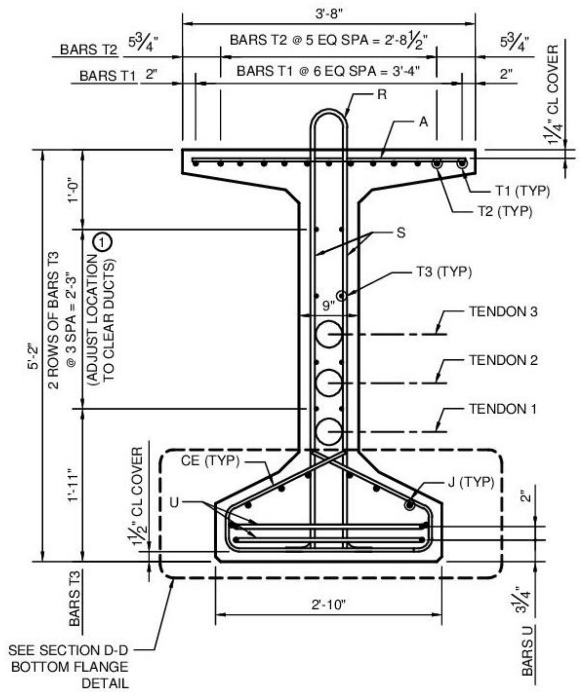
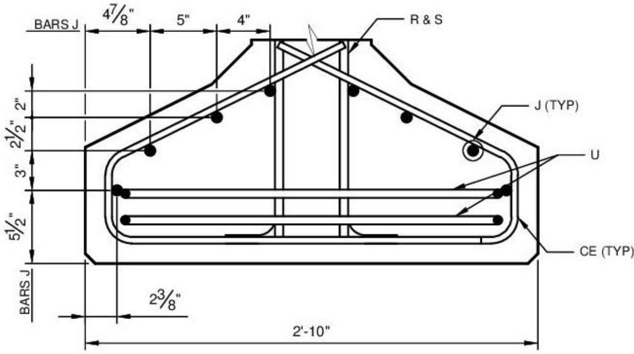
SEGMENT B BEAM END DETAILS

R-3



SECTION D-D

(PRESTRESSING NOT SHOWN)



SECTION D-D BOTTOM FLANGE DETAIL

(PRESTRESSING NOT SHOWN)

GENERAL NOTES:

1. TENDON LOCATIONS MAY VARY FROM THOSE SHOWN

① PLACEMENT OF BARS T3 SHOULD MATCH THAT OF ADJACENT GIRDER SEGMENT

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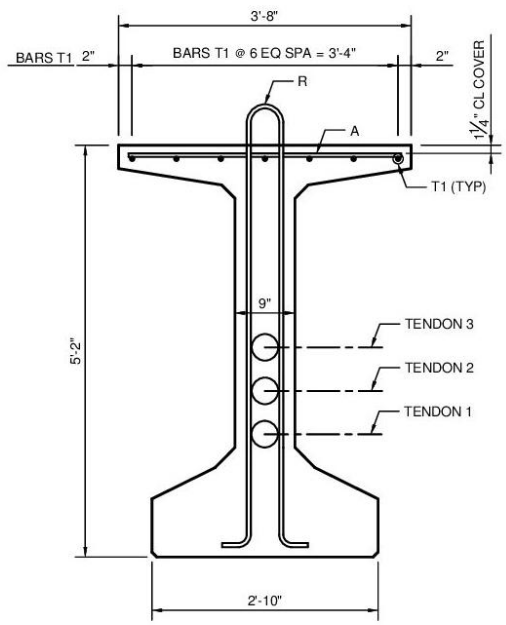
SPLICE REGION DETAILS

TxDOT SPLICED GIRDER PROJECT

Project #: 0-6652

SECTION D-D

R-4



TYPICAL SECTION OUTSIDE OF END REGION

(PRESTRESSING NOT SHOWN)

GENERAL NOTES:

1. TENDON LOCATIONS MAY VARY FROM THOSE SHOWN

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SPLICE REGION DETAILS

TxDOT SPLICED GIRDER PROJECT

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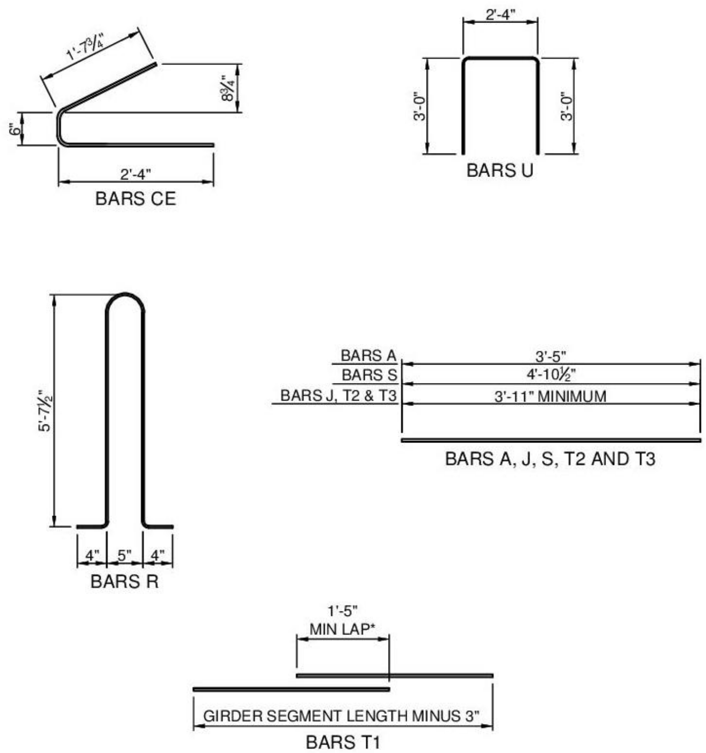
TYPICAL SECTION

R-5

REINFORCING STEEL	
BAR	SIZE
A	3
CE	4
J	6
R	5
S	-
T1	4
T2	5
T3	4
U	5



* LAP BARS WHERE NECESSARY ALONG LENGTH OF GIRDER SEGMENT

GENERAL NOTES

1. SHEAR REINFORCEMENT IN THE SPLICE REGION SHALL BE THE LARGER OF THAT PROVIDED IN THE ADJACENT PRECAST GIRDER SEGMENTS
2. ENSURE CLEAR COVER REQUIREMENTS TO TRANSVERSE REINFORCEMENT ARE SATISFIED WITH SPECIFIED COMBINATION OF WEB WIDTH AND DUCT DIAMETER. SIZE OF DUCT COUPLERS SHOULD ALSO BE CONSIDERED.

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SPLICE REGION DETAILS

TxDOT SPLICED GIRDER PROJECT

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REINFORCING DETAILS AND GENERAL NOTES

R-6

Appendix E. Proposed Modifications to the AASHTO LRFD (2014) General Shear Procedure

INTRODUCTION

The proposed modifications to the AASHTO LRFD (2014) general shear procedure that were developed during the first phase of the spliced girder research program are presented in this appendix. Details leading to the development of the proposed provisions are provided in Chapter 2. The provisions were first introduced in Moore (2014) and are repeated below. Please refer to Moore (2014) for further information.

PROPOSED SHEAR DESIGN PROCEDURE

Based on the findings of the spliced girder research program, the modifications to the general shear procedure of Article 5.8.3.4.2 of AASHTO LRFD (2014) outlined below are proposed. The recommended changes can be summarized by the following two points:

(i) The reduction in the effective web width, b_v, specified in Article 5.8.2.9 to account for the presence of a post-tensioning duct should no longer be applied to the general shear procedure. In its place, the shear resistance provided by the transverse reinforcement, V_s, should be multiplied by the shear strength reduction factor, λ_{duct} (defined below). With the use of the proposed shear strength reduction factor, the limit on the duct diameter specified in Article 5.4.6.2 can be increased with respect to the general shear procedure.

(ii) The gross web width, b_w, should replace the effective web width, b_v, within the expressions for the nominal shear resistance, V_n, and the shear resistance provided by the concrete, V_c, in Article 5.8.3.3 when using the general shear procedure. (Further research may be needed to evaluate the upper limit placed on V_n.)

Implementation of these proposed revisions results in the following shear design procedure (modifications to AASHTO LRFD (2014) are shown in bold):

The shear strength contribution provided by tensile stresses in the concrete, V_c, and the contribution provided by the shear reinforcement, V_s, are calculated as follows (modified from Article 5.8.3.3 of AASHTO LRFD (2014)):

$$V_c = 0.0316\beta\sqrt{f'_c}\,\boldsymbol{b_w} d_v \tag{E.1}$$

$$V_s = \frac{\boldsymbol{\lambda_{duct}} A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \tag{E.2}$$

where:

A_v = Area of shear reinforcement within distance s (in.^{2})

b_w = Gross web width (in.)

d_e = Effective depth measured from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement (in.)

d_v = Effective shear depth equal to the distance measured perpendicular to the neutral axis between the resultants of the tensile and compressive forces due to flexure, not to be taken as less than the greater of $0.9d_e$ or $0.72h$ (in.)

f'_c = Specified compressive strength of concrete (ksi)

f_y = Specified minimum yield strength of reinforcement (ksi)

h = Overall member depth (in.)

s = Spacing of transverse reinforcement measured in the direction parallel to the longitudinal reinforcement (in.)

α = Angle of inclination of transverse reinforcement to the longitudinal axis (degrees)

β = Factor indicating ability of diagonally cracked concrete to transmit tension and shear

θ = Angle of inclination of diagonal compressive stresses (degrees)

λ_{duct} = Shear strength reduction factor to account for the effect of the presence of a post-tensioning duct in the member's web

The shear strength reduction factor, λ_{duct}, is calculated as follows:

$$\boldsymbol{\lambda_{duct} = 1 - \delta\left(\frac{\emptyset_{duct}}{b_w}\right)^2} \qquad (E.3)$$

where:

δ = Duct diameter correction factor, taken as 2.0 for grouted plastic and steel ducts

$\emptyset_{duct}$ = Diameter of post-tensioning duct in the member's web within depth d_v (in.)

The factors β and θ may be determined from the expressions provided below (from Article 5.8.3.4.2 of AASHTO LRFD (2014)):

For sections containing at least the minimum amount of transverse reinforcement:

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \qquad (E.4)$$

For all sections:

$$\theta = 29 + 3500\varepsilon_s \tag{E.5}$$

For the section under consideration, the net longitudinal tensile strain at the centroid of the tension reinforcement, ε_s, may be calculated as follows (from Article 5.8.3.4.2 of AASHTO LRFD (2014)):

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + \left|V_u - V_p\right| - A_{ps}f_{po}\right)}{E_sA_s + E_pA_{ps}} \tag{E.6}$$

where:

A_{ps} = Area of prestressing steel on the flexural tension side of the member (in.^{2})

A_s = Area of nonprestressed steel on the flexural tension side of the member (in.^{2})

E_p = Modulus of elasticity of prestressing strands (ksi)

E_s = Modulus of elasticity of reinforcing bars (ksi)

f_{po} = A parameter taken as the modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi)

$|M_u|$ = Absolute value of the factored moment, not to be taken as less than $|V_u - V_p|d_v$ (kip-in.)

N_u = Factored axial force, taken as positive if tensile and negative if compressive (kip)

V_p = Component of the effective prestressing force in the direction of the applied shear, taken as positive if resisting the applied shear (kip)

V_u = Factored shear force (kip)

The nominal shear resistance, V_n, is then calculated as the lesser of the following expressions (modified from Article 5.8.3.3 of AASHTO LRFD (2014)):

$$V_n = V_c + V_s + V_p \tag{E.7}$$

$$V_n = 0.25f'_c \boldsymbol{b_w} d_v + V_p \tag{E.8}$$

Appendix F. Data from Experimental Tests

INTRODUCTION

Data collected from the instrumentation monitored during the spliced girder tests and the shear-friction push-through tests are provided in this appendix. Data from the girder tests are presented first followed by the push-through test data.

SPLICED GIRDER TEST DATA

Linear potentiometer and foil strain gauge data from the two cast-in-place (CIP) splice region tests are presented in this section. For both tests, the total shear force acting at the critical section (refer to Section 5.2.2) is plotted against the strain or linear potentiometer measurements. The total shear force includes the effects of the girder self-weight and the weight of the load frame, as described in Section 5.2.2.

LINEAR POTENTIOMETER DATA

Linear Potentiometers at the Splice Region

Measurements collected from the linear potentiometers located in the vicinity of the CIP splice regions of the test girders (see Figure F1) are presented below. The specific placement of the linear potentiometers is shown in Figure F2. The nomenclature used to identify each potentiometer in the plots is then defined in Figure F3. Lastly, based on this nomenclature, each sensor is labeled in Figure F4. For the vertically oriented linear potentiometers attached to the web of the girder, positive displacement values correspond with the shearing motion indicated by the arrows at the splice region interfaces in Figure F4. For the linear potentiometers mounted on floor stands (labeled as Floor V(N), Floor V(M), and Floor V(S)), the plots presented in Section 5.4.1 are repeated. A description of how these plots were created can be found in that section. Please note that not all the sensors provided complete data up to the maximum shear force, V_{test}. In these cases, the plots terminate before reaching the value of V_{test} (V_{test} = 666 kips for Girder 1 and 703 kips for Girder 2).

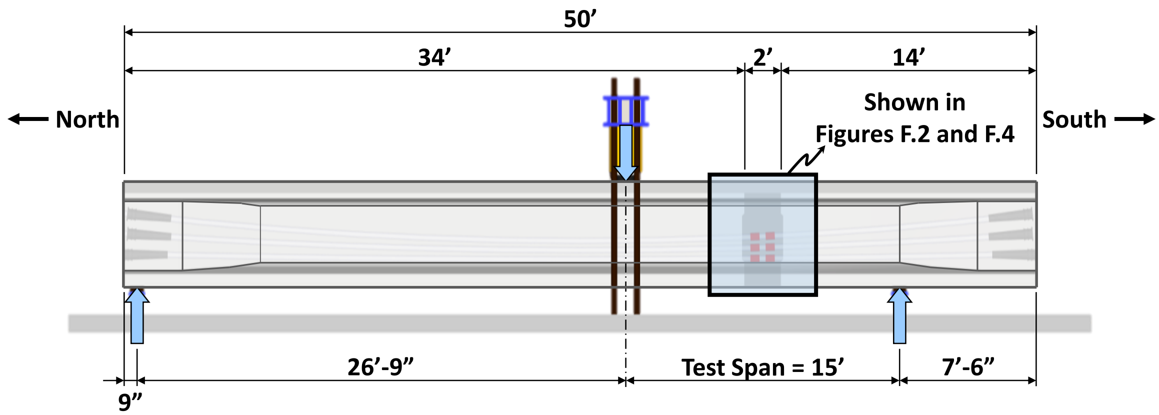


Figure F1: General location of linear potentiometers installed at the splice region

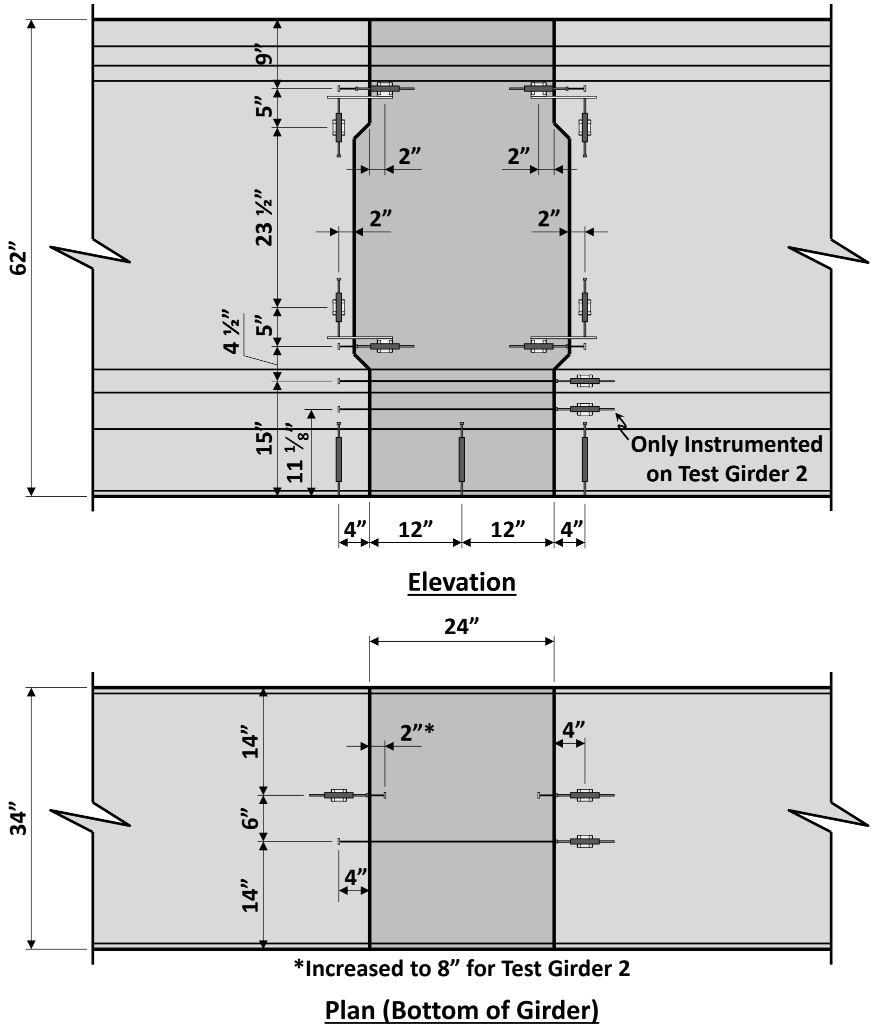


Figure F2: Placement of linear potentiometers at the splice region

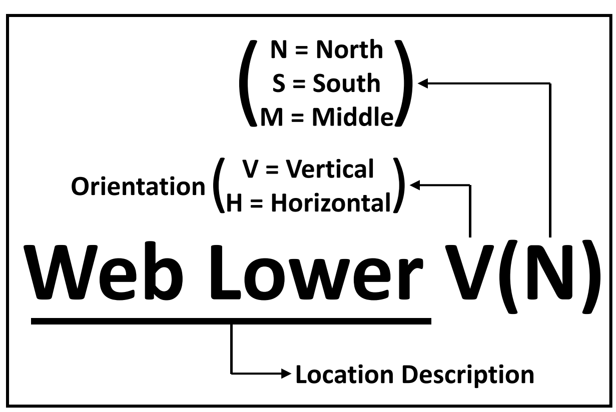


Figure F3: Nomenclature for linear potentiometers at the splice region

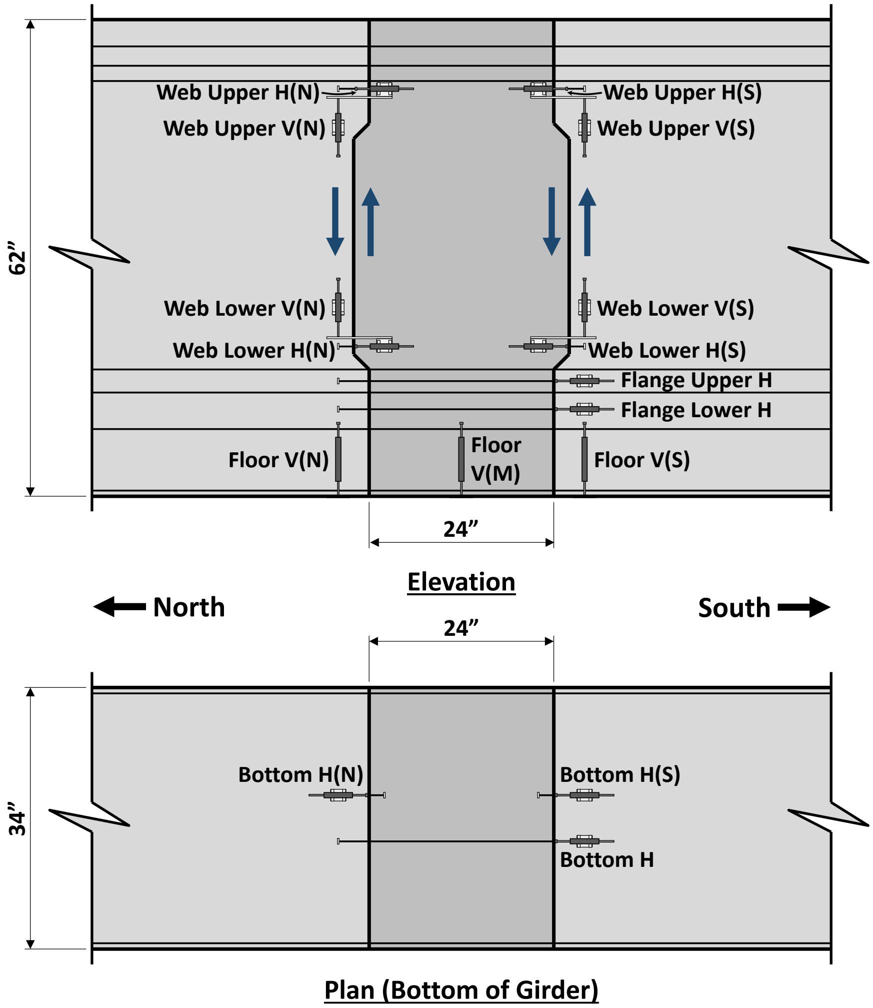
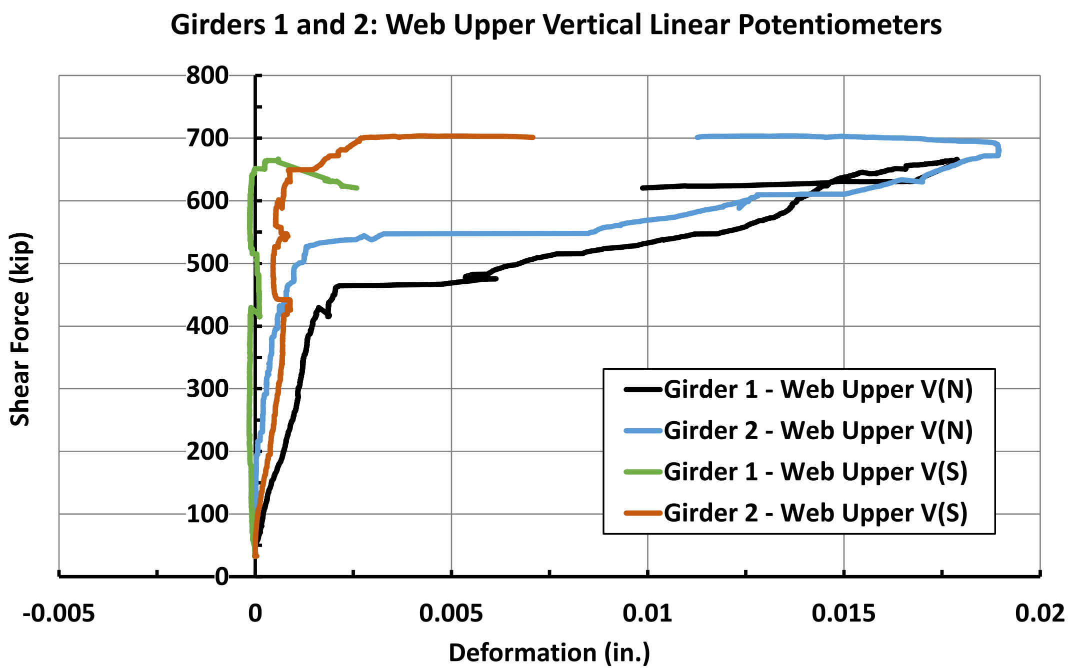
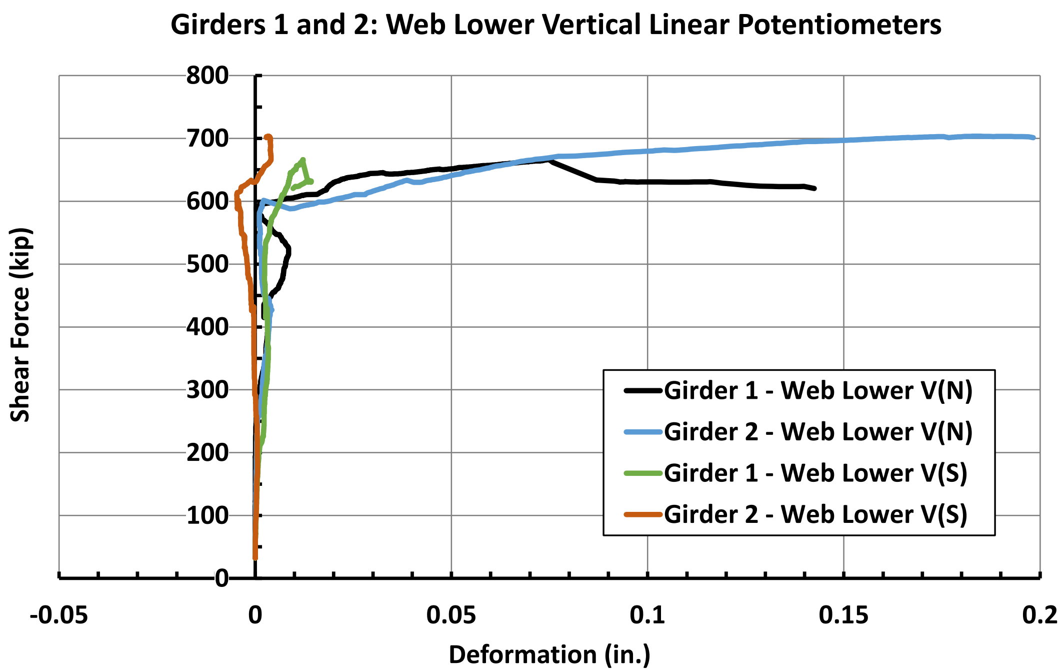


Figure F4: Designations of linear potentiometers at the splice region

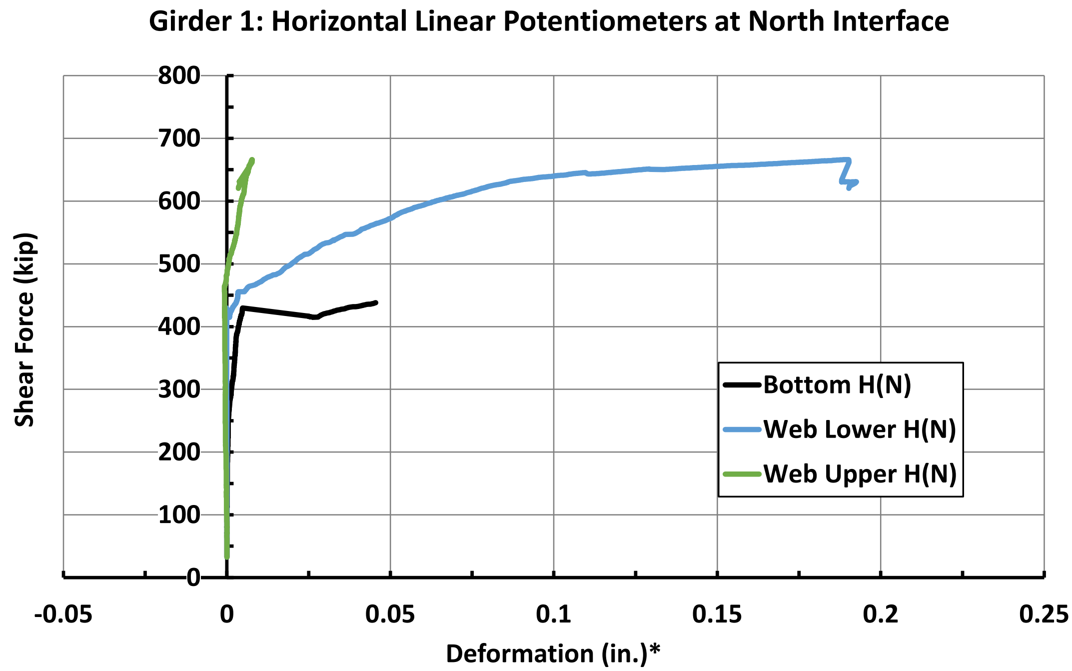


(a)

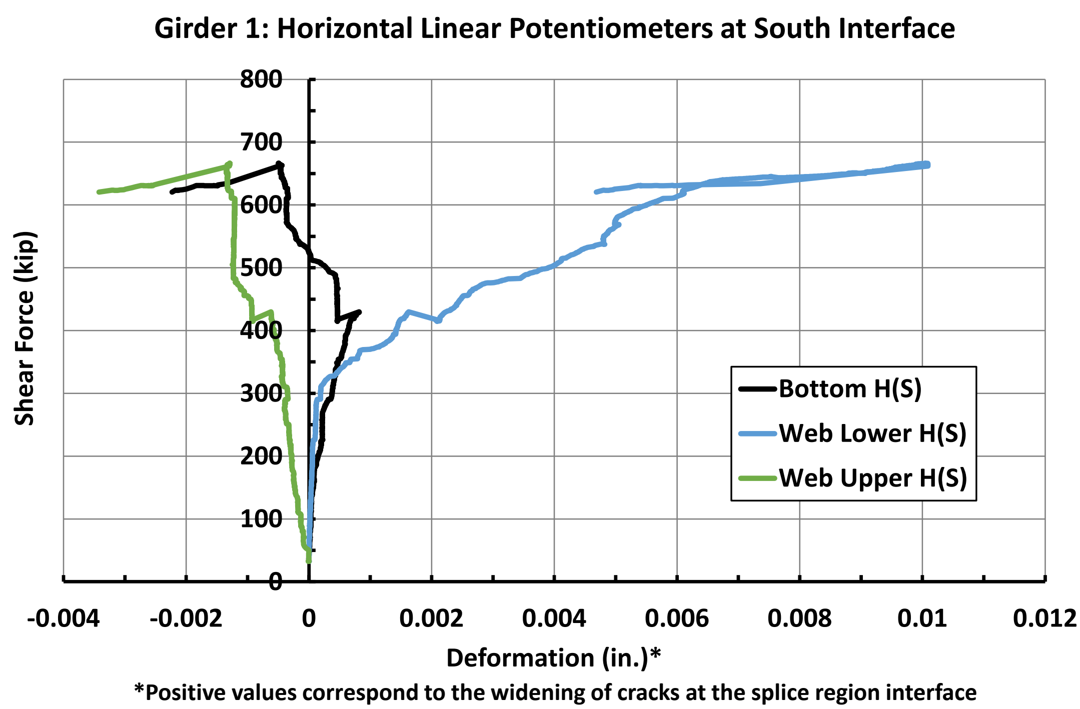


(b)

Figure F5: Vertical linear potentiometers mounted on web – Girders 1 and 2

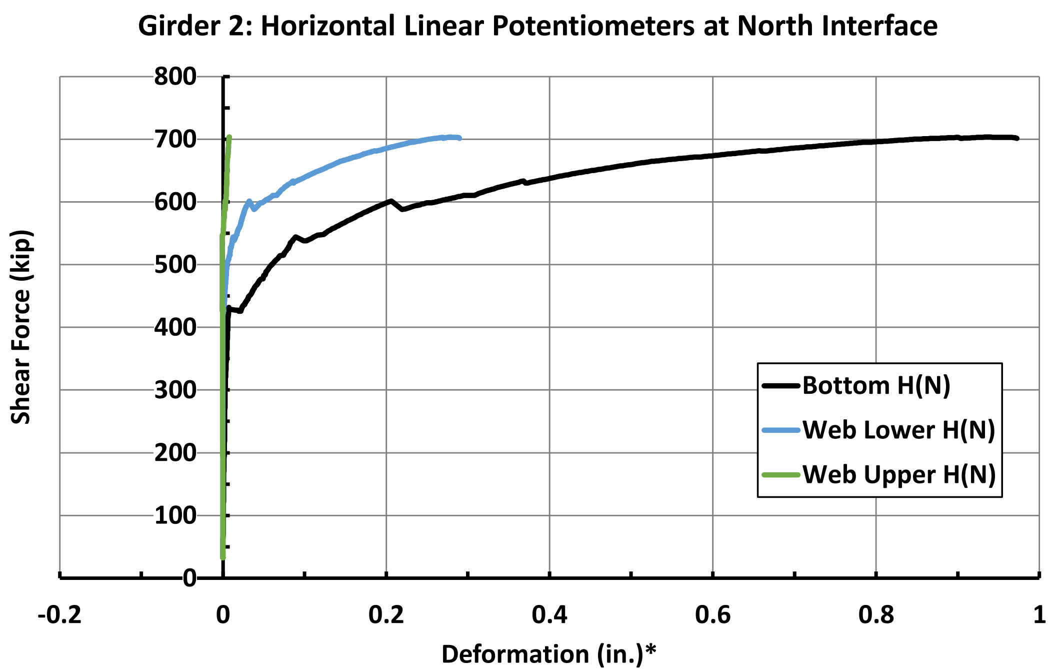


(a)

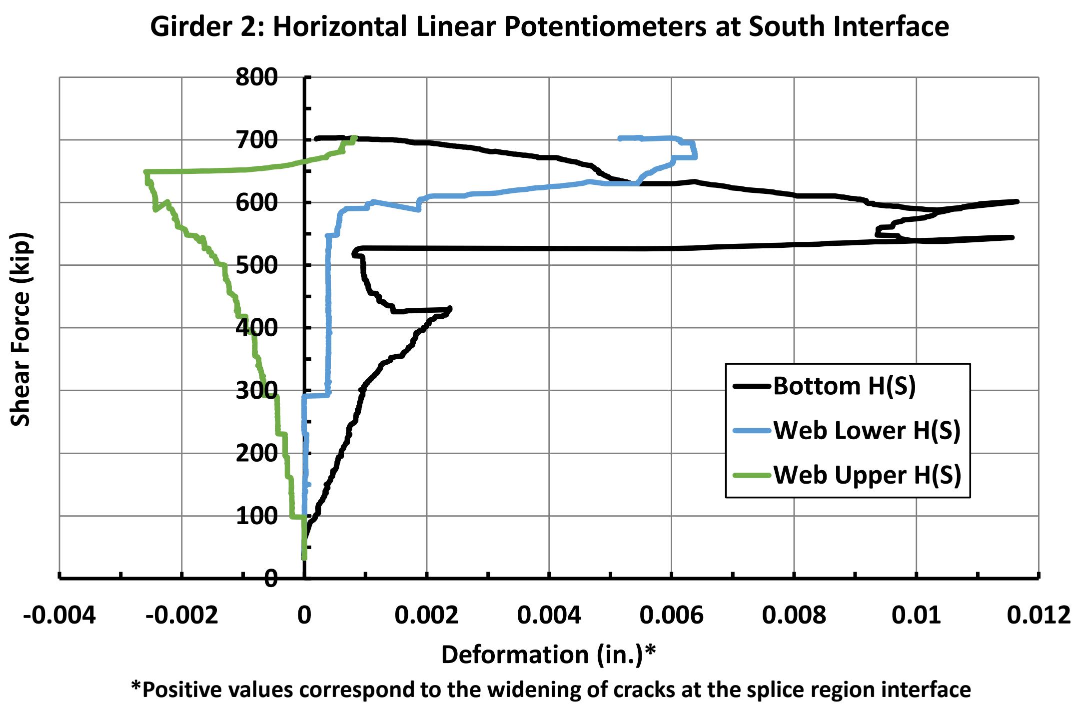


(b)

Figure F6: Horizontal linear potentiometers at splice region interfaces – Girder 1

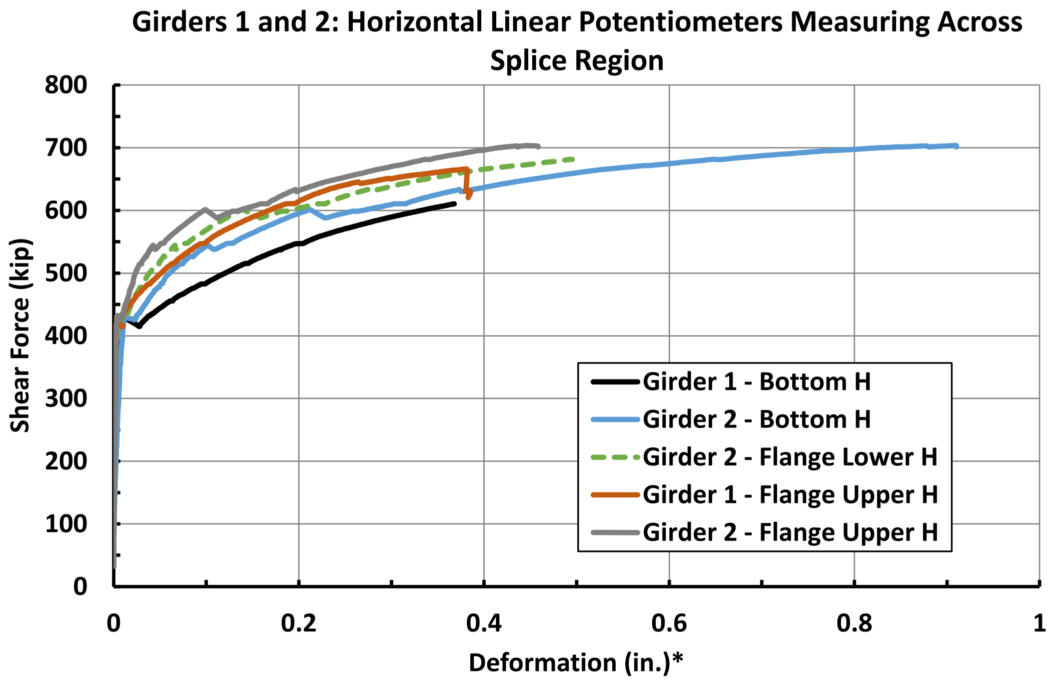


(a)

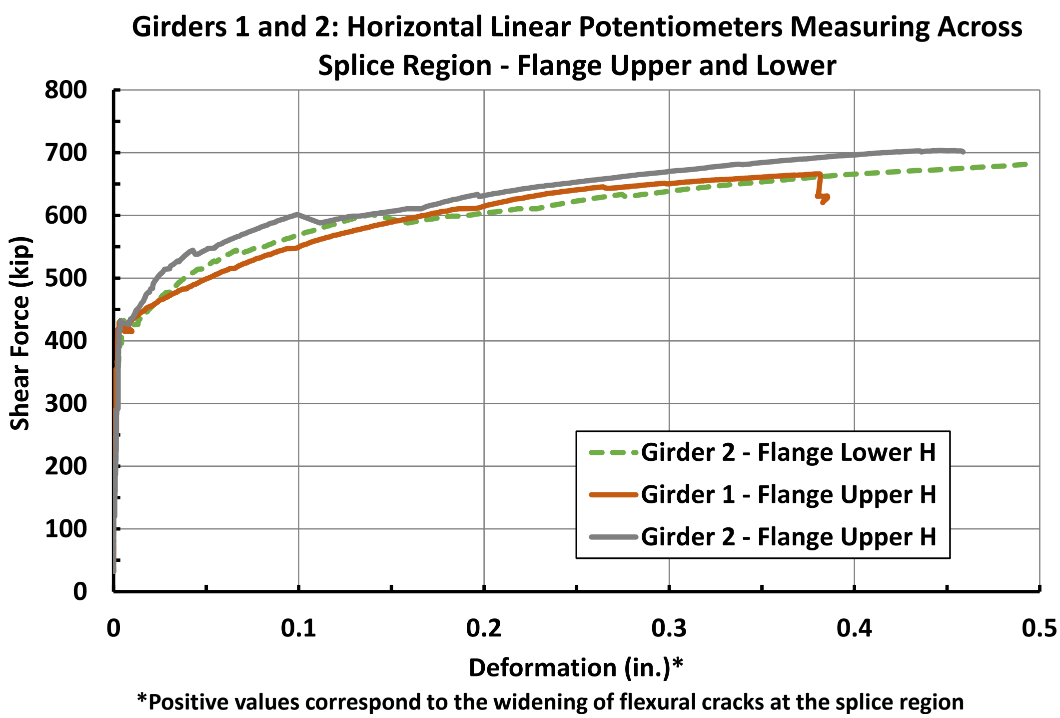


(b)

Figure F7: Horizontal linear potentiometers at splice region interfaces – Girder 2

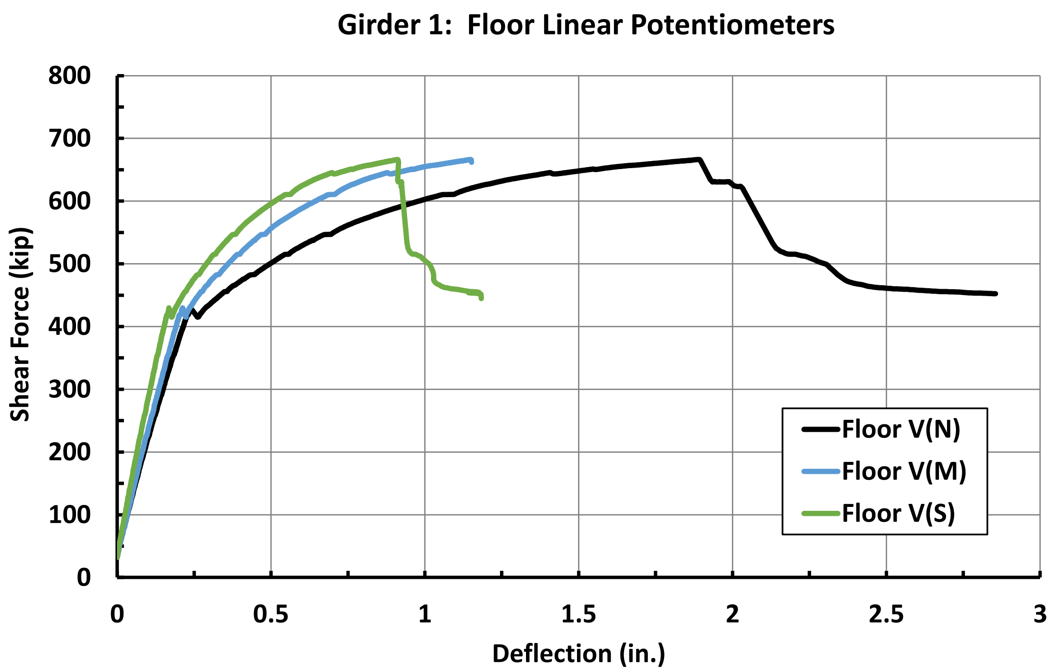


(a)

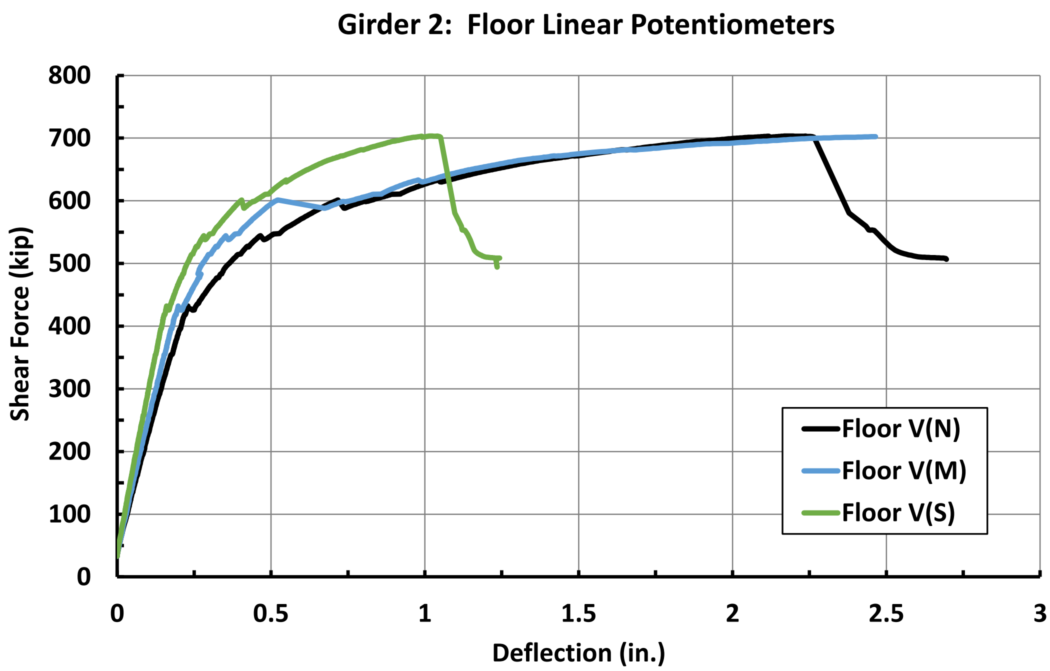


(b)

Figure F8: Horizontal linear potentiometers measuring across splice region – Girders 1 and 2



(a)



(b)

Figure F9: Girder deflections measured by linear potentiometers at the splice region – Girders 1 and 2

Deflection at the Load Point

The shear force at the critical section is plotted against the deflection measured under the load point for both test girders in Figures F10 and F11 below. The plots are repeated from Section 5.2.3. The deflection measurements were obtained by averaging the output from two linear potentiometers located on opposite sides of the girders at the location of the load point. Rigid body motion indicated by displacements measured at the supports was subtracted from the deflection values.

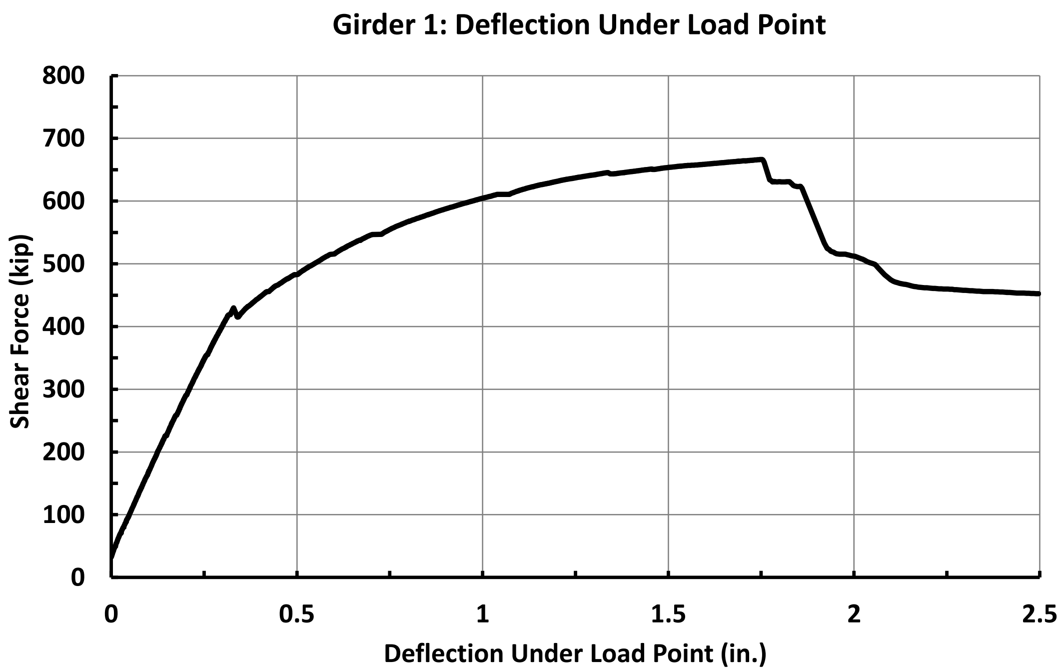


Figure F10: Girder deflection under the load point – Girder 1

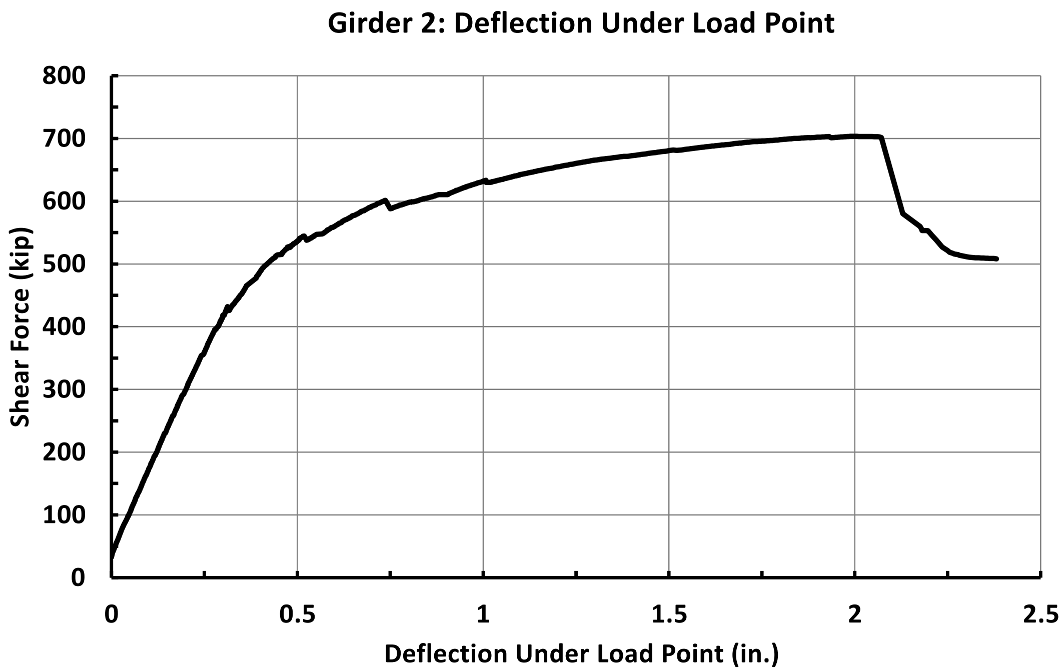


Figure F11: Girder deflection under the load point – Girder 2

FOIL STRAIN GAUGE DATA

Data from the foil strain gauges installed on the mild reinforcement within the splice regions of the test girders are presented in this section. It should be noted that many of the strain gauges failed to produce accurate readings as the ultimate load was approached, possibly due to debonding or other damage to the gauges or their lead wires. For this reason, the strain data could not be plotted up to the maximum shear force in several cases.

Strain Gauges Installed on Interface Reinforcement

Data from the strain gauges installed on the longitudinal interface reinforcement extending from the precast segments into the CIP splice regions are presented below. All gauges were located within the splice regions of the test girders, not the precast segments. The nomenclature used to identify each strain gauge in the plots is defined in Figure F12.

The placement of each gauge on the interface reinforcement is indicated in Figures F13 and F14 for Test Girder 1 and Test Girder 2, respectively. A majority of the strain gauges were installed near the faces of the precast segments, as pictured in Figure F15(a). However, five gauges indicated by the abbreviation “Mid” in Figures F13 and F14 were installed at the middle of the splice regions (i.e., 12 in. from the faces of the precast segments, as shown in Figure F15(b)). Please note that an initial compressive strain in the reinforcement that was introduced and monitored during the post-tensioning operations is included in the plots.

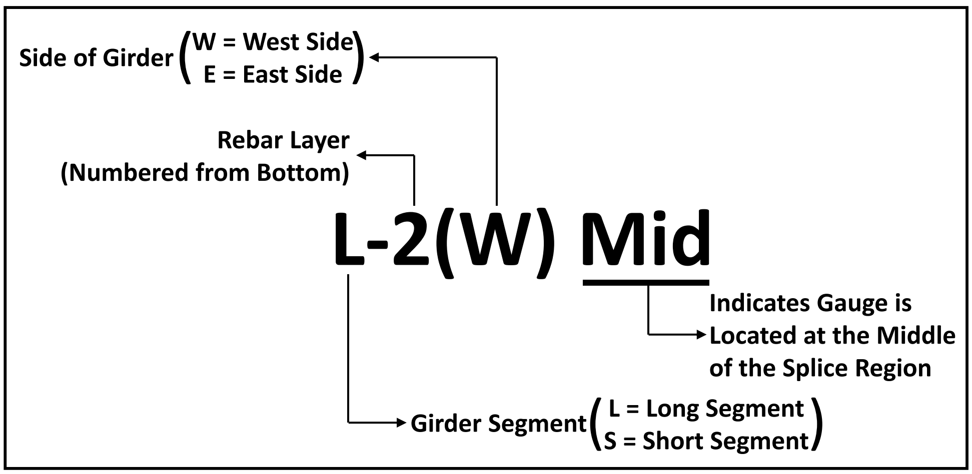


Figure F12: Nomenclature for strain gauges installed on interface reinforcement

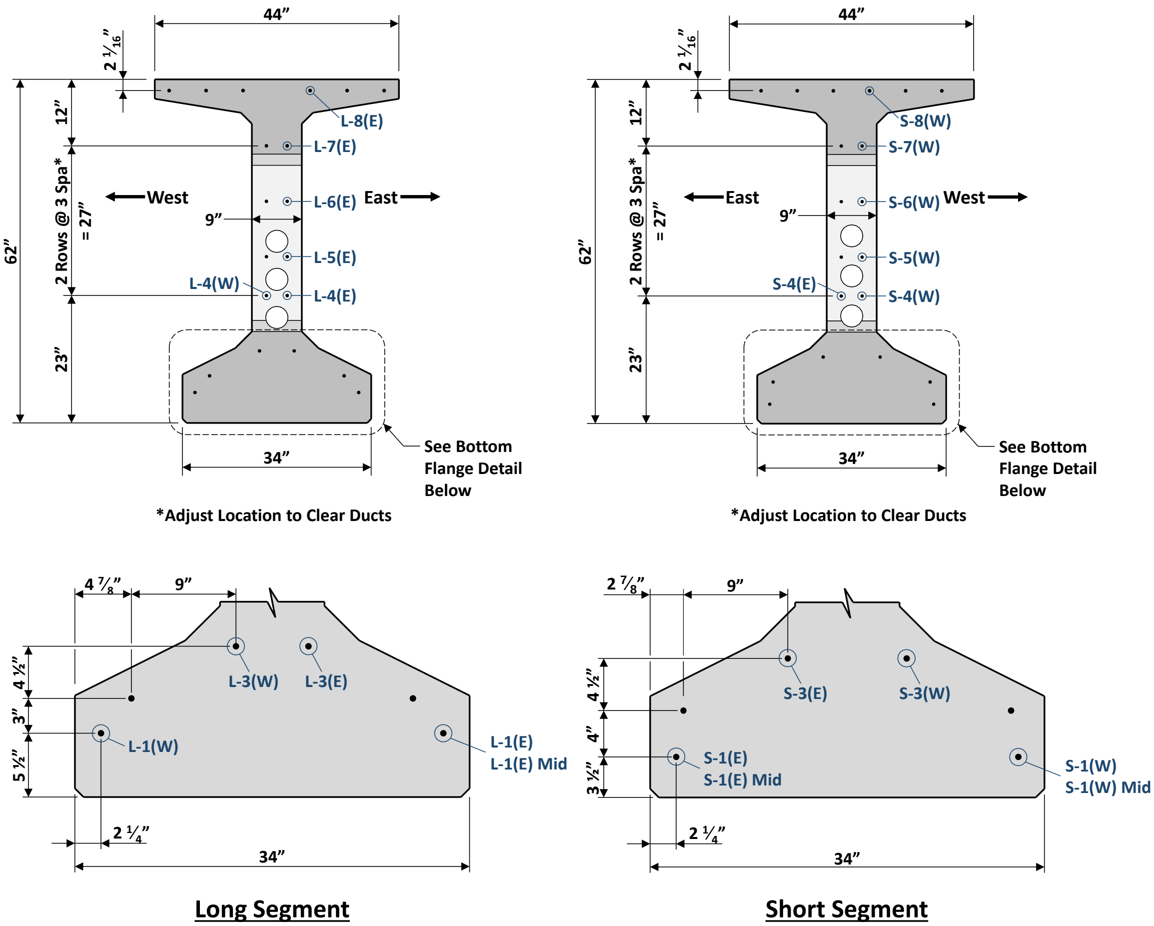


Figure F13: Designations of strain gauges installed on interface reinforcement – Girder 1

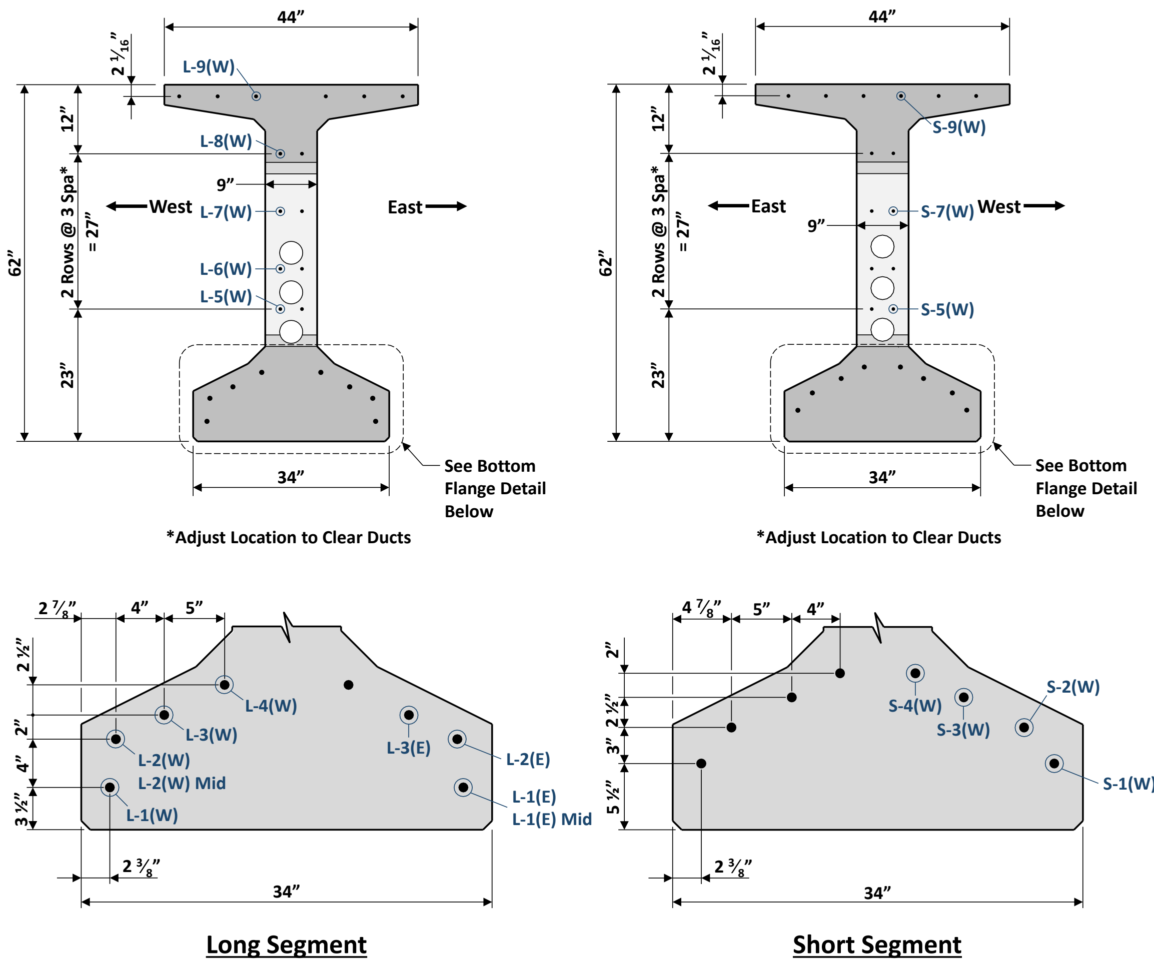
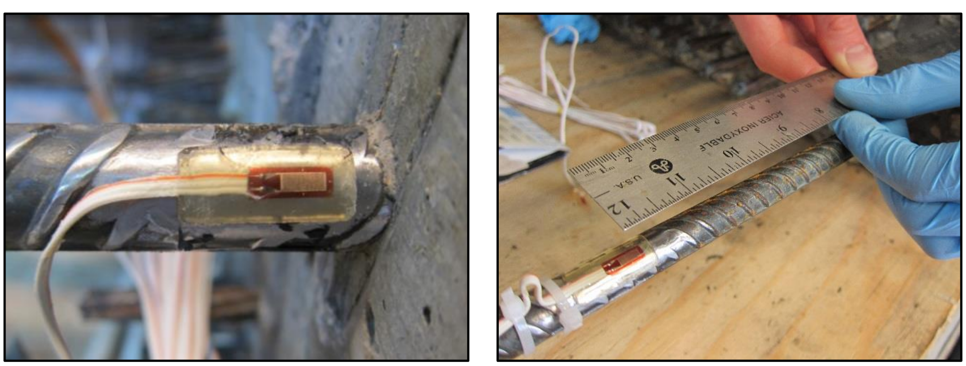
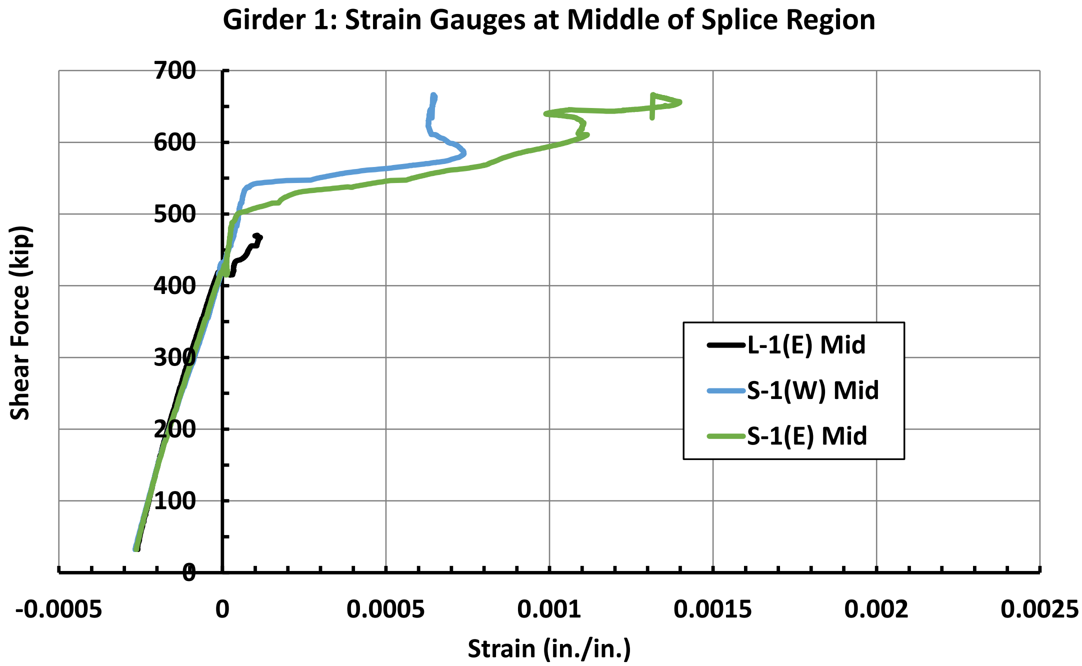


Figure F14: Designations of strain gauges installed on interface reinforcement – Girder 2

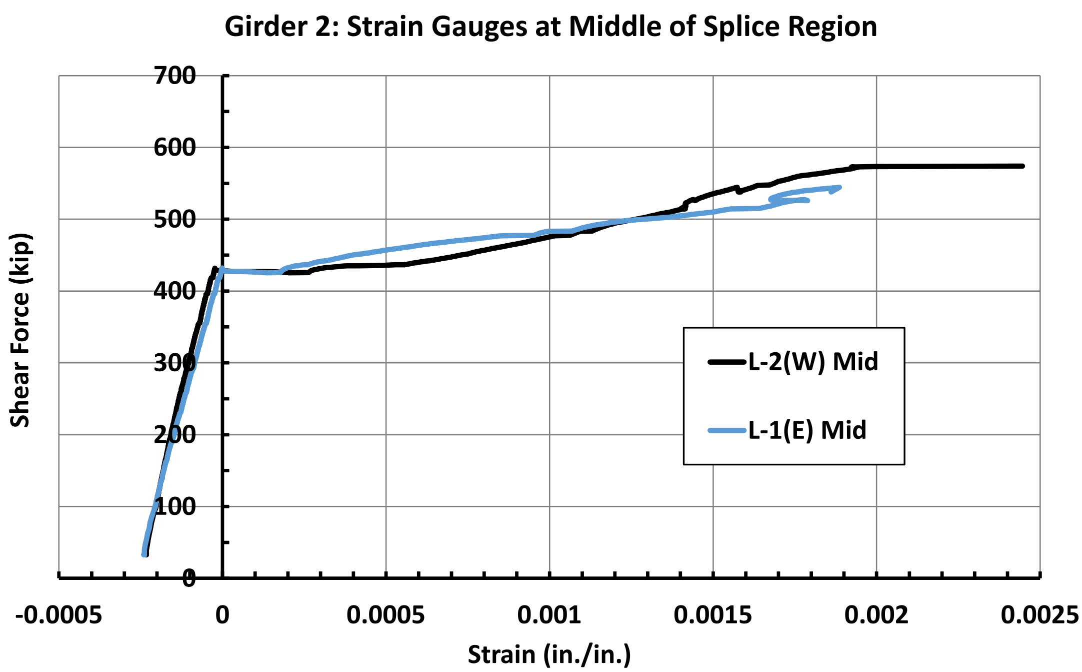


(a) *(b)*

Figure F15: Strain gauge placement – (a) at face of precast segment; (b) at middle of splice region



(a)



(b)

Figure F16: Strain gauges at the middle of the splice region – Girders 1 and 2

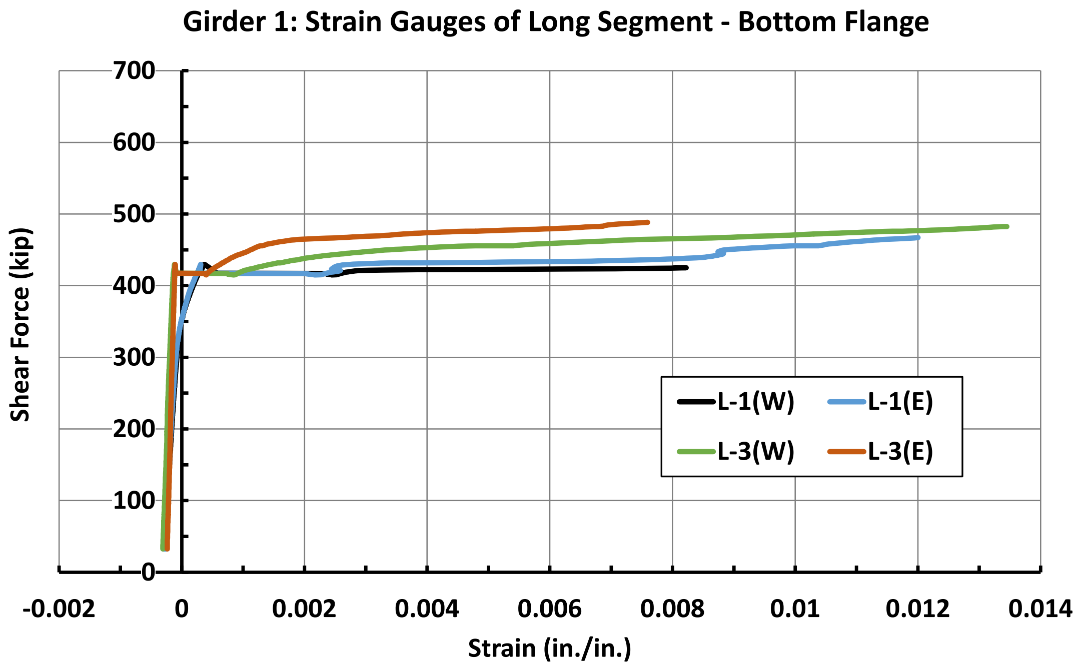
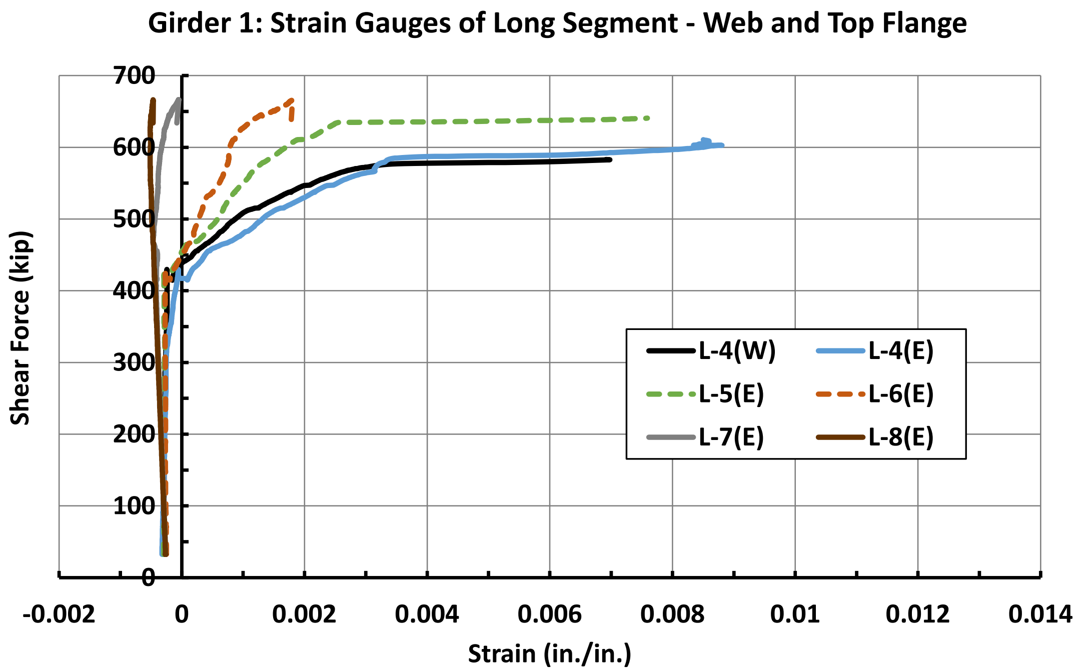
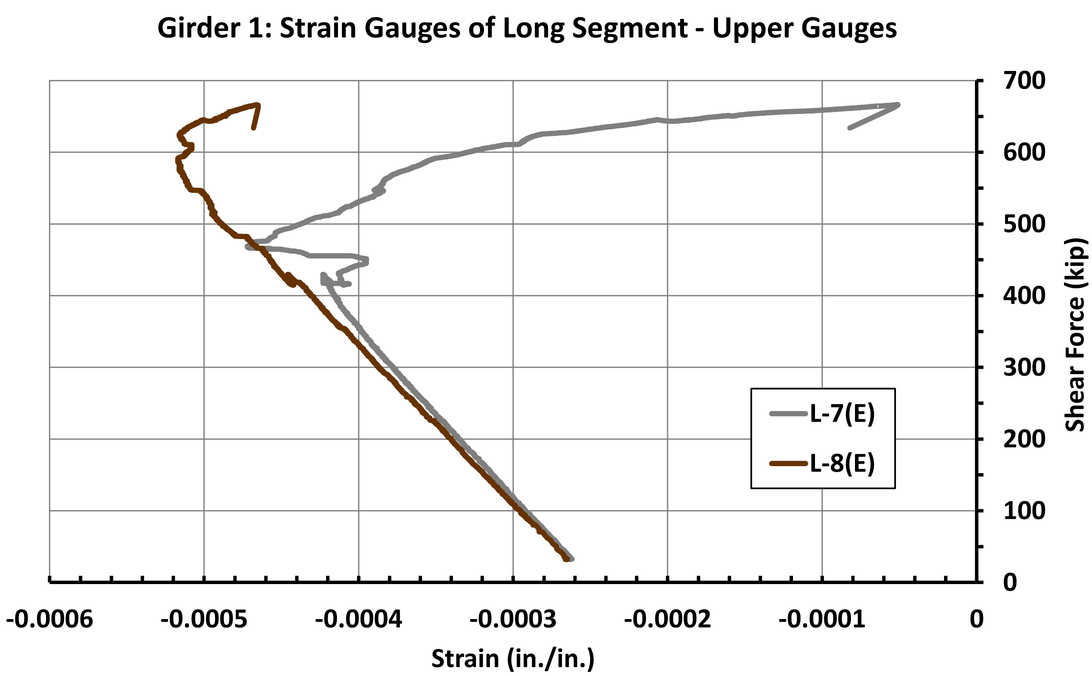


Figure F17: Strain gauges in bottom flange of long precast segment – Girder 1

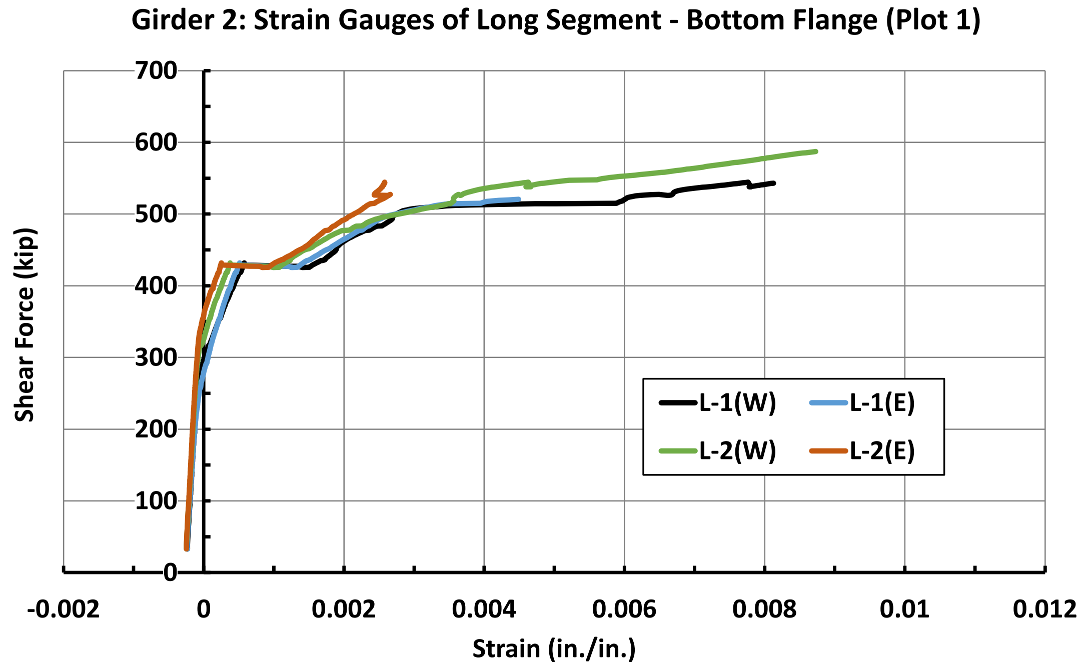


(a)

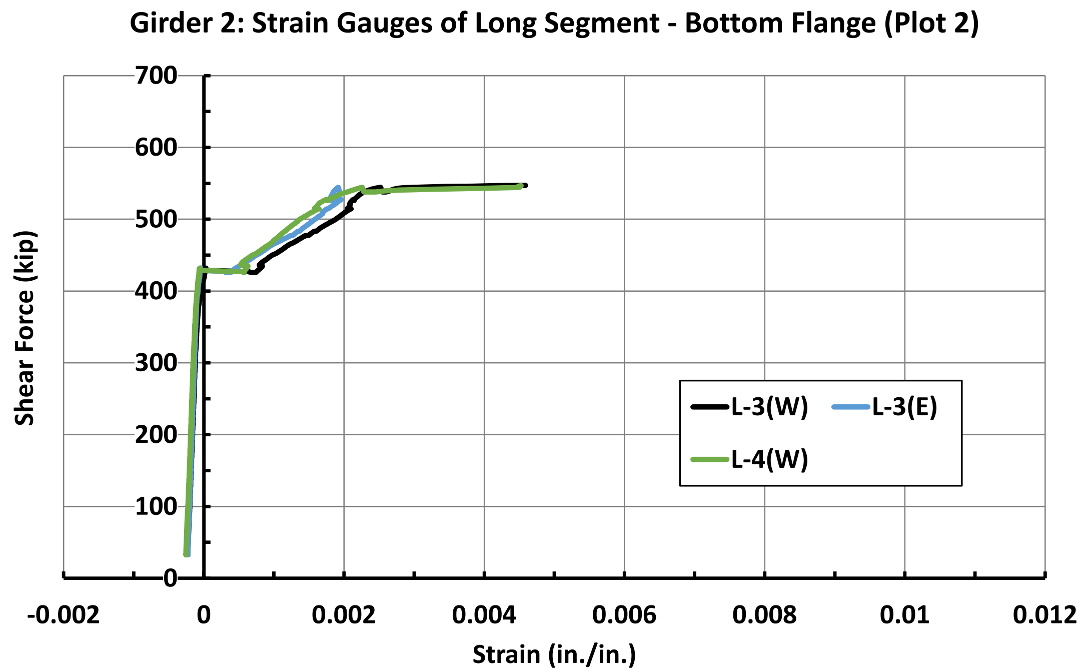


(b)

Figure F18: Strain gauges in web and top flange of long precast segment – Girder 1

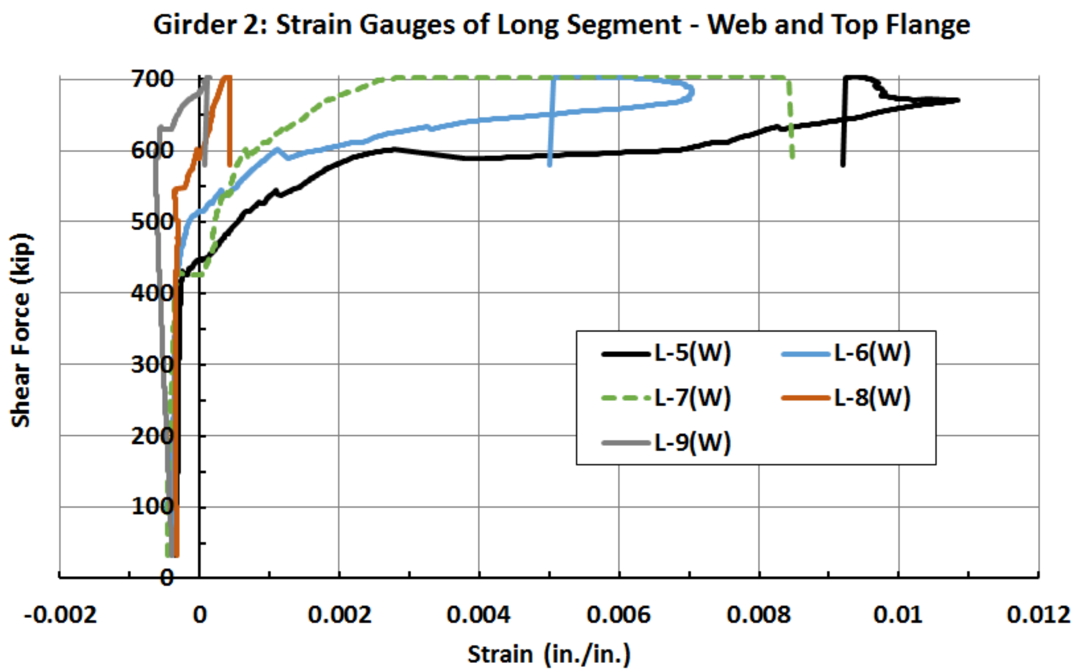


(a)

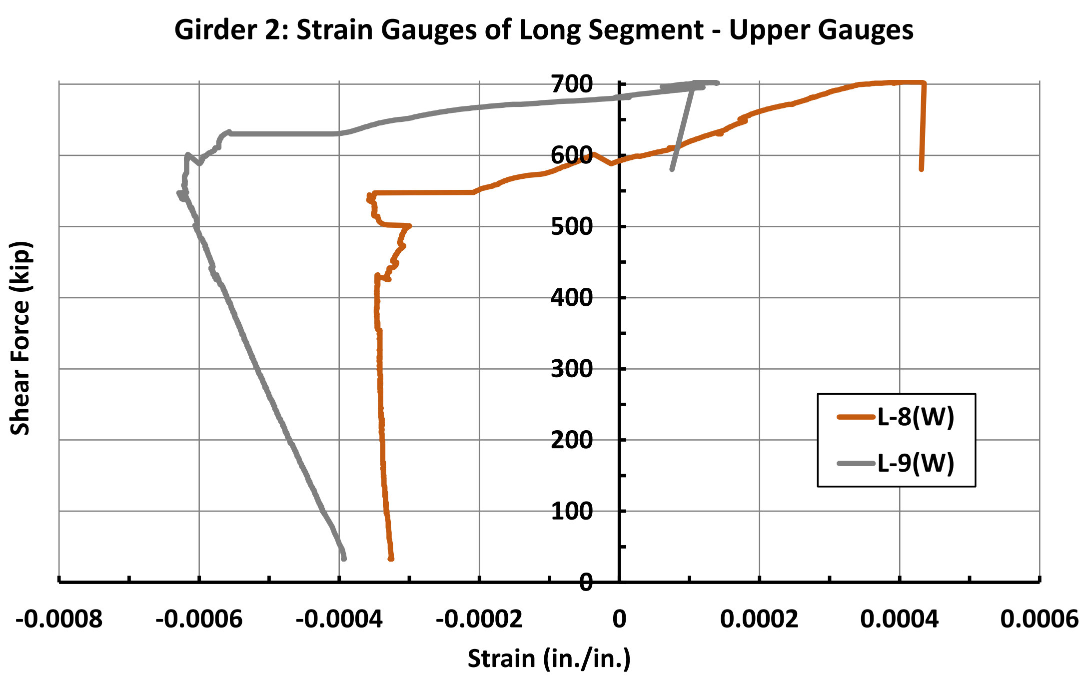


(b)

Figure F19: Strain gauges in bottom flange of long precast segment – Girder 2



(a)



(b)

Figure F20: Strain gauges in web and top flange of long precast segment – Girder 2

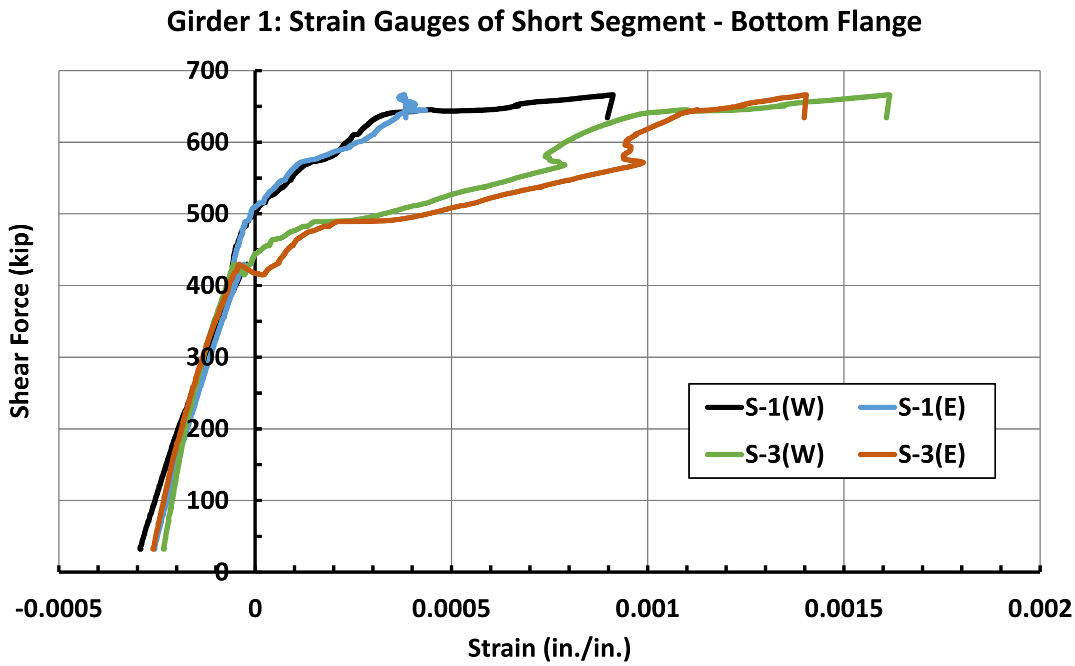
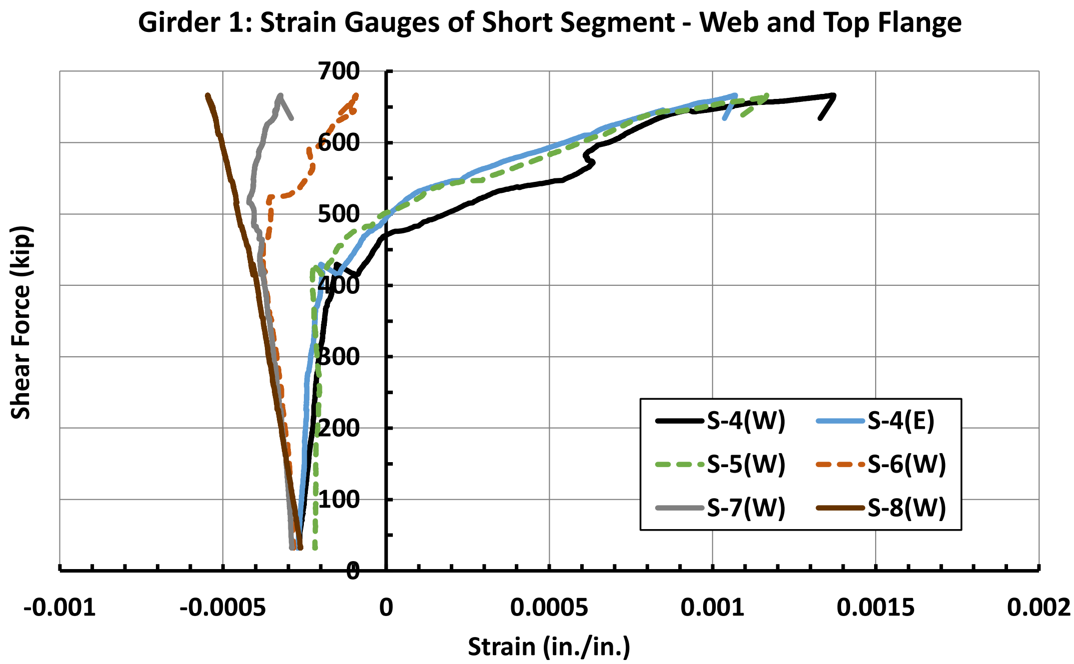
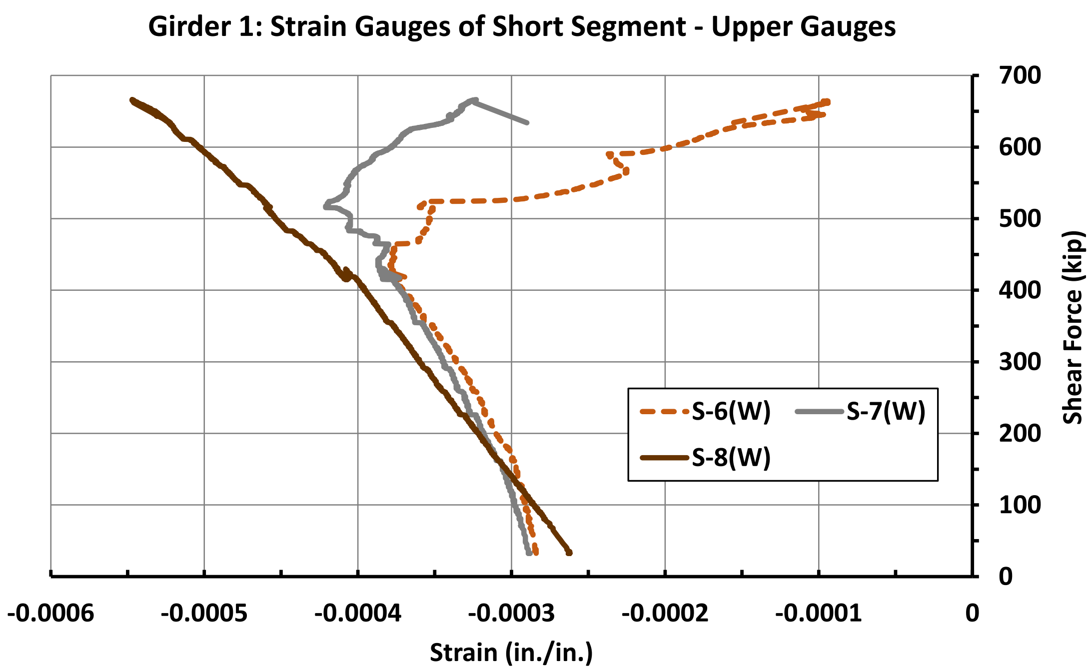


Figure F21: Strain gauges in bottom flange of short precast segment – Girder 1

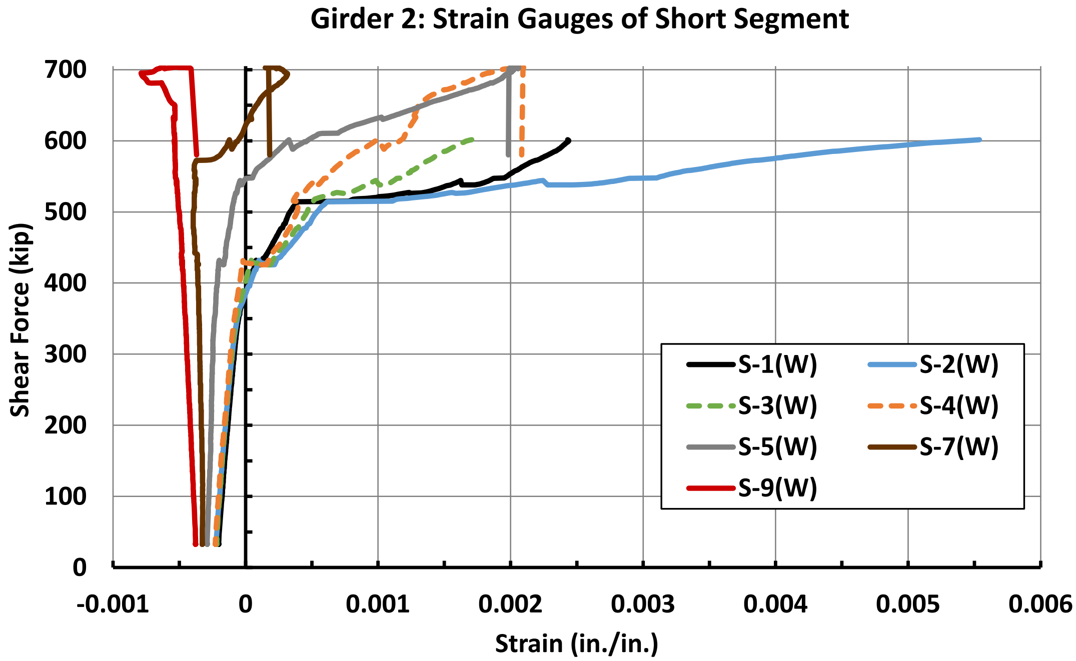


(a)

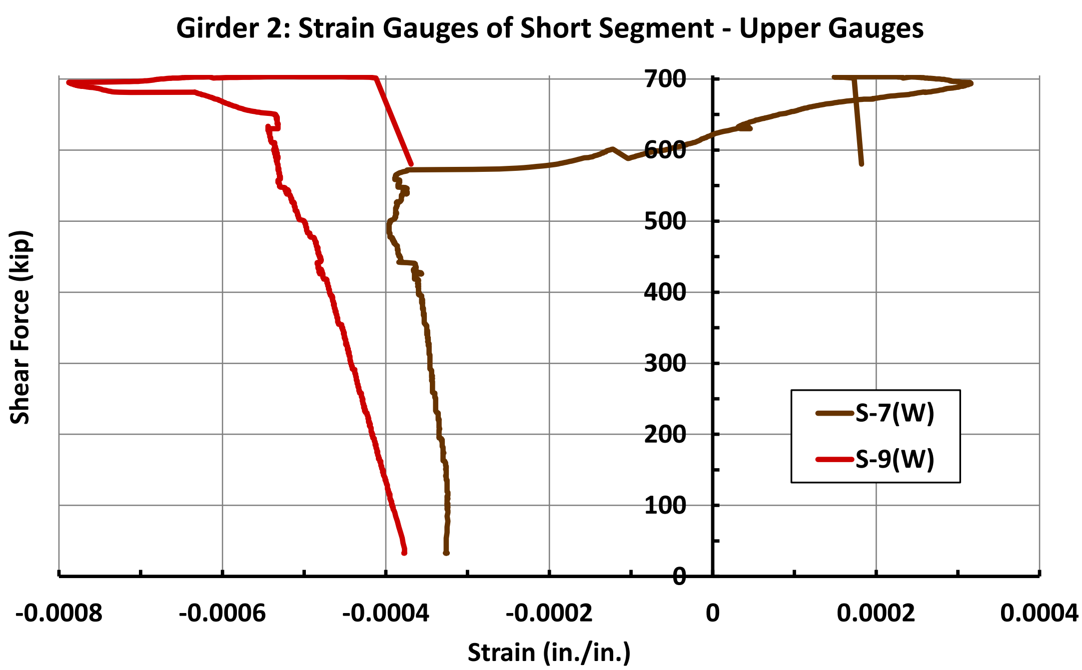


(b)

Figure F22: Strain gauges in web and top flange of short precast segment – Girder 1



(a)



(b)

Figure F23: Strain gauges of short precast segment – Girder 2

Strain Gauges Installed on Stirrups

The two stirrups located at the center of the CIP splice region of each girder were instrumented with strain gauges as illustrated in Figure F24. The placement of the gauges along the reinforcing bars at Section A-A and Section B-B is indicated in the figure. At each section, three strain gauges were placed to correspond with the locations of the post-tensioning ducts. A fourth strain gauge was also installed in Test Girder 2 to monitor rebar strains above the ducts. The strain data from each gauge are plotted below.

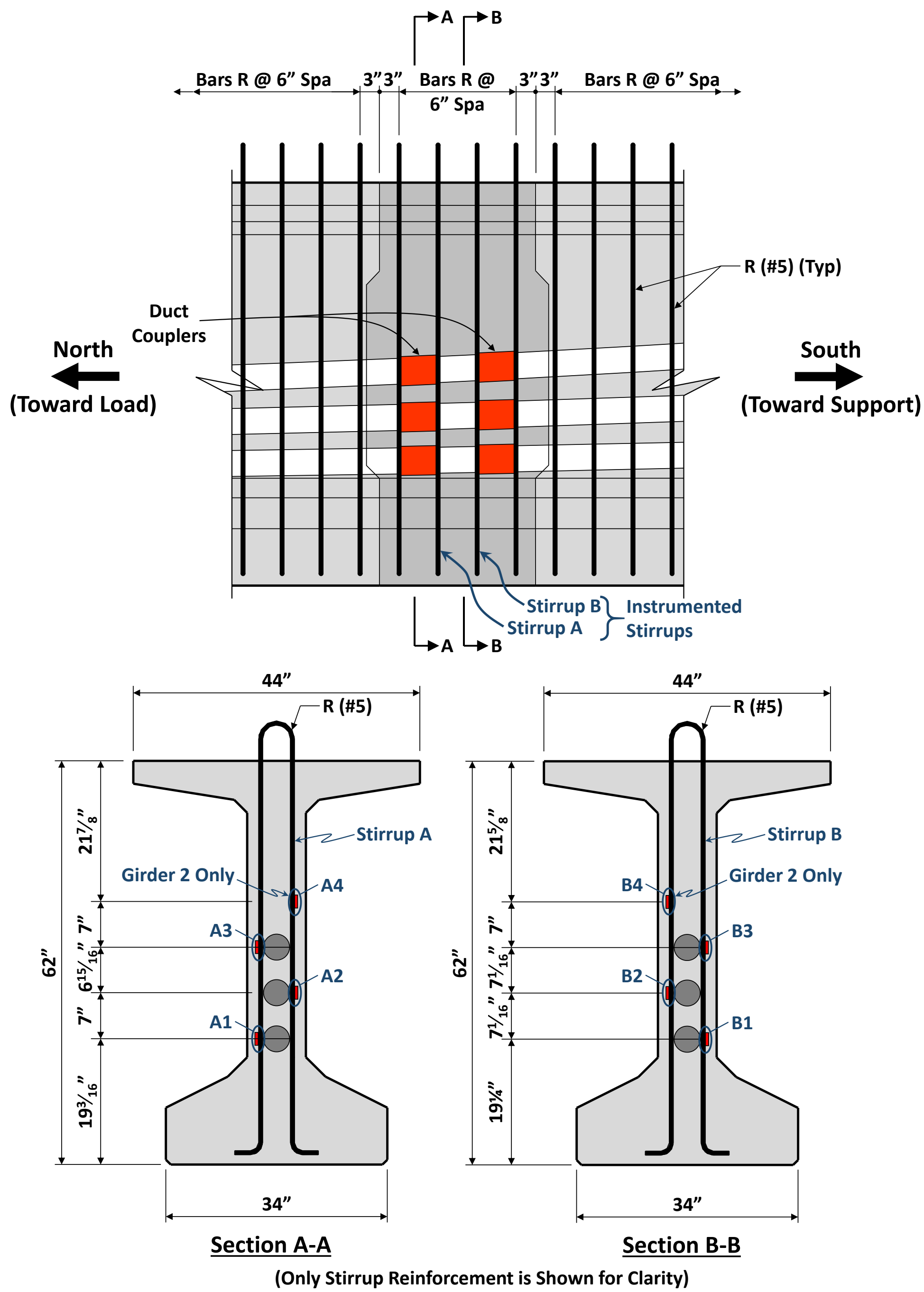
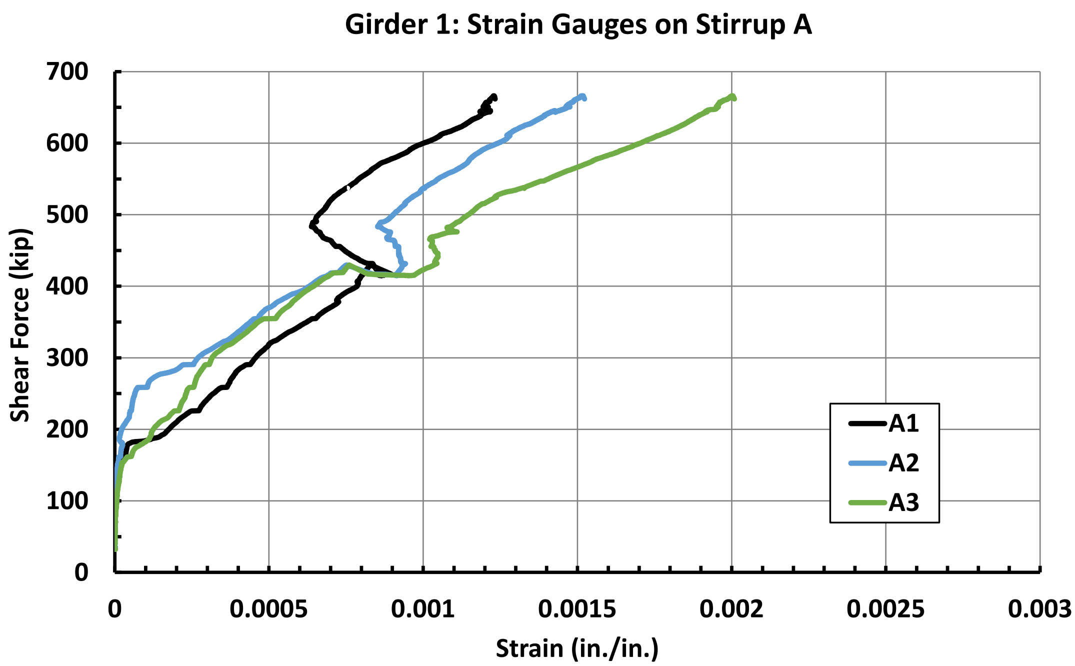
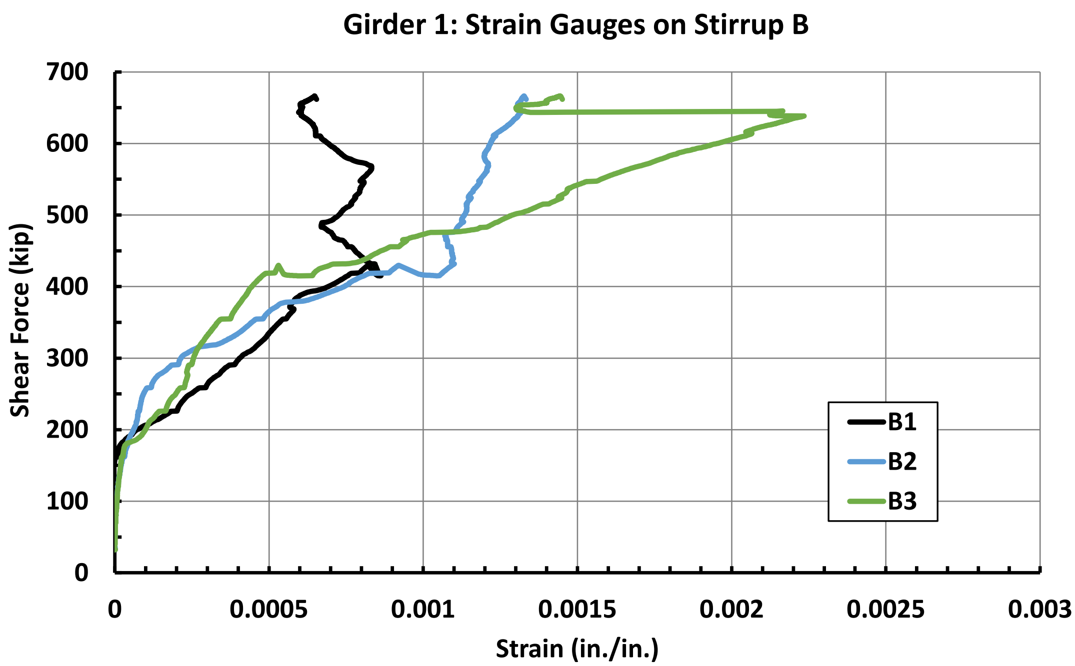


Figure F24: Designations of strain gauges installed on stirrup reinforcement

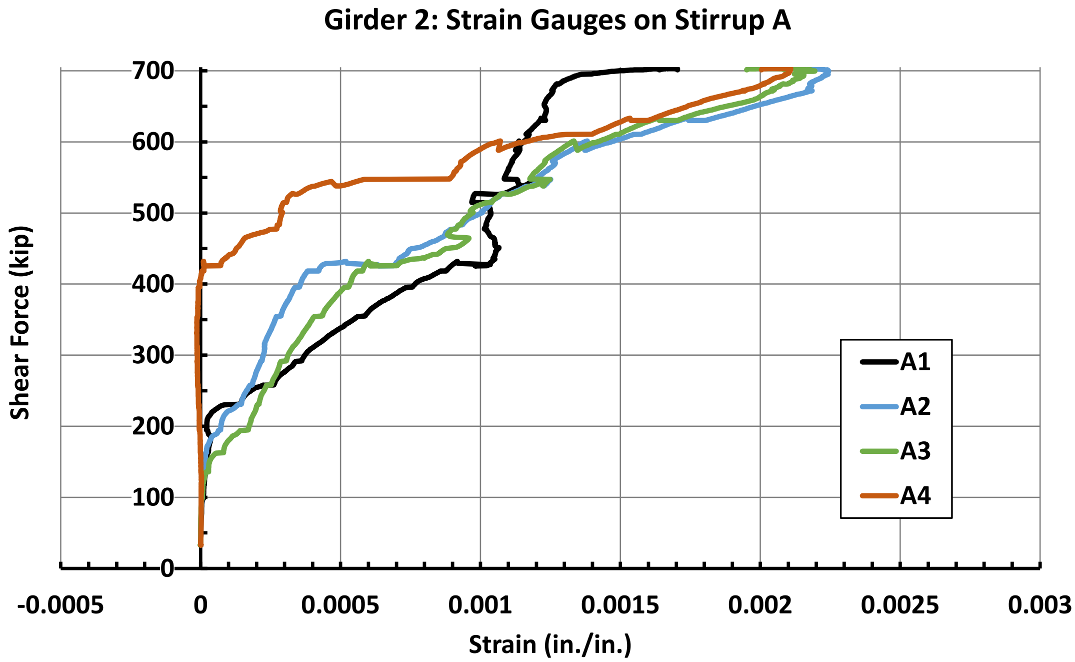


(a)

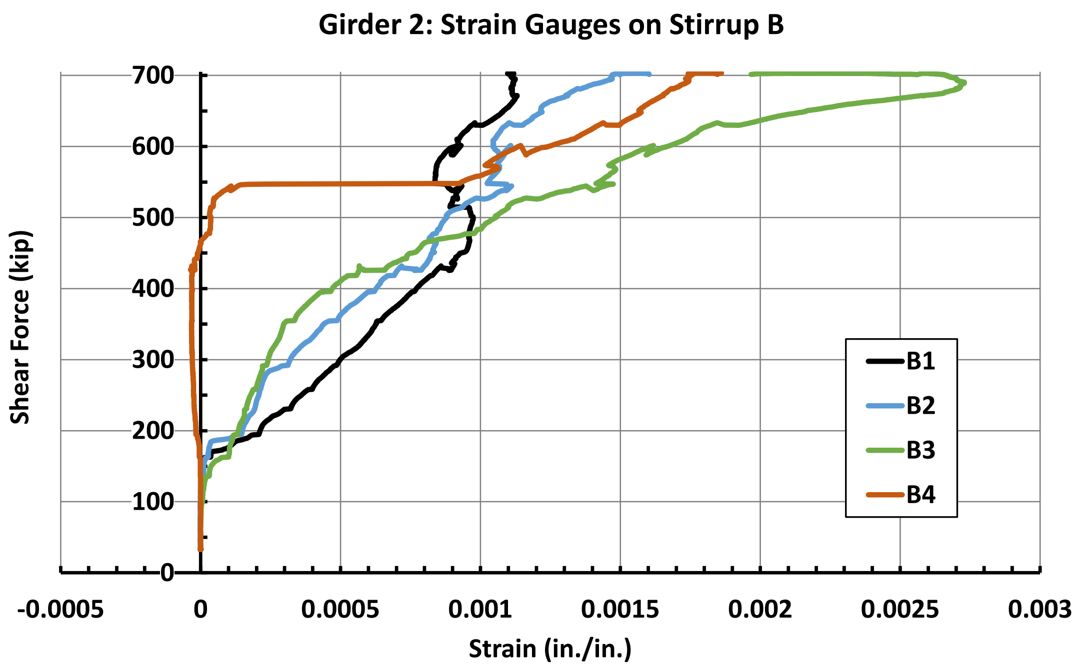


(b)

Figure F25: Strain gauges on the stirrup reinforcement in the splice region – Girder 1



(a)



(b)

Figure F26: Strain gauges on the stirrup reinforcement in the splice region – Girder 2

SHEAR-FRICTION TEST DATA

Data from linear potentiometers installed on the push-through test specimens of the shear-friction experimental program are provided in this section. For each sensor, the shear force acting along the interface (refer to Section 6.3.1) is plotted against the corresponding displacement data. The sensors are referred to as "top" and "bottom" linear potentiometers as indicated in Figure F27. The graphs are divided based on whether the displacements were measured across the failure interface or not (refer to Section 6.3.2). The other shear interface is referred to as the "non-failure" interface. The data are provided up to a measured displacement of 0.1 in. For three of the eleven tests, a linear potentiometer did not function properly. Displacement data corresponding to these sensors are therefore not presented. Additional information concerning the linear-potentiometer instrumentation is provided in Section 6.2.4.

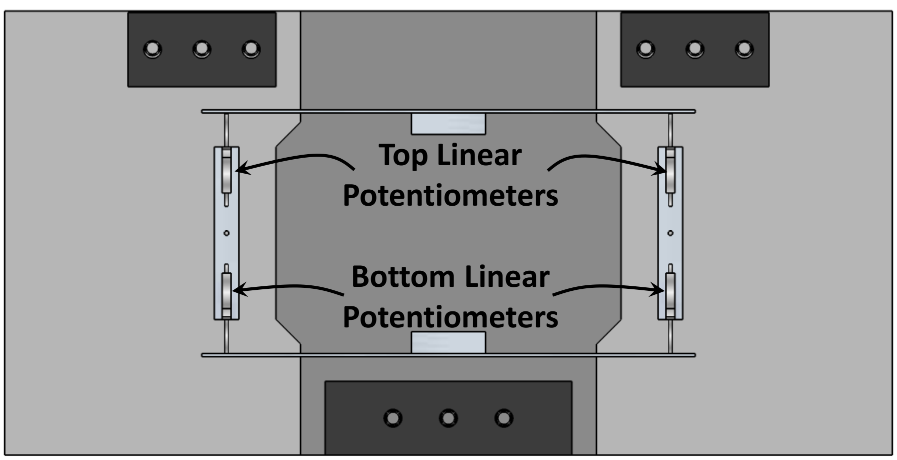
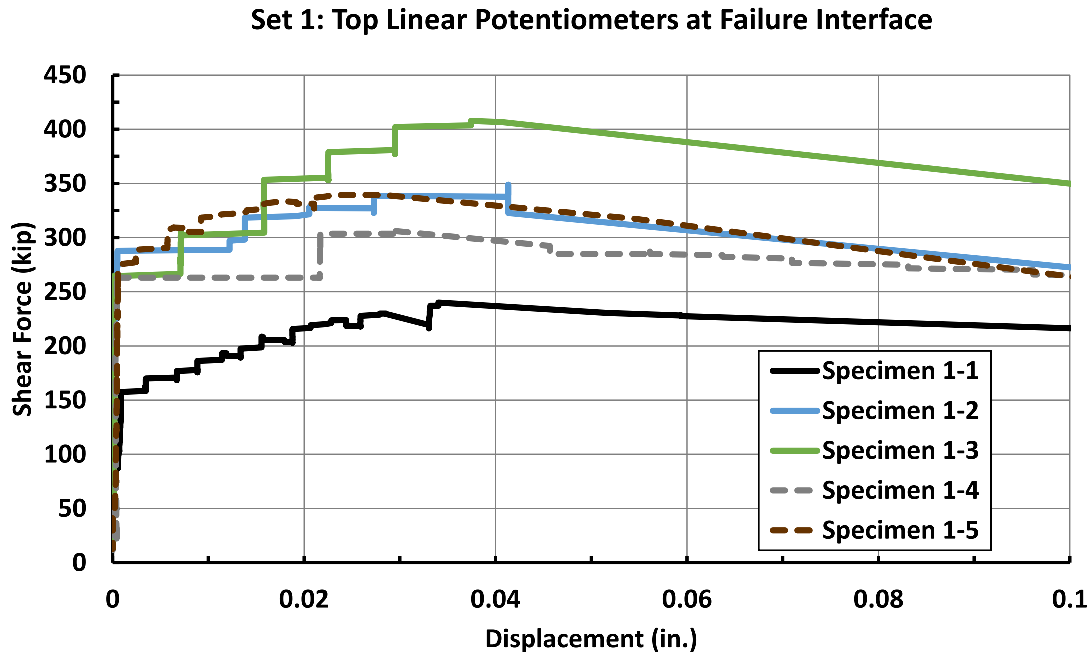
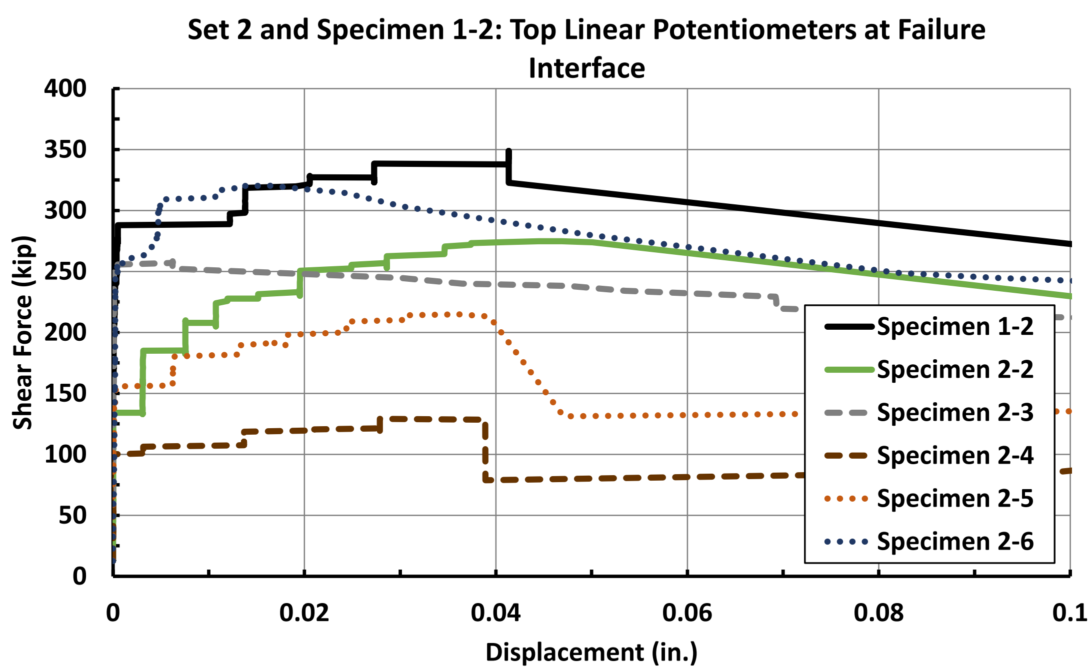


Figure F27: Designations of linear potentiometers on push-through specimens

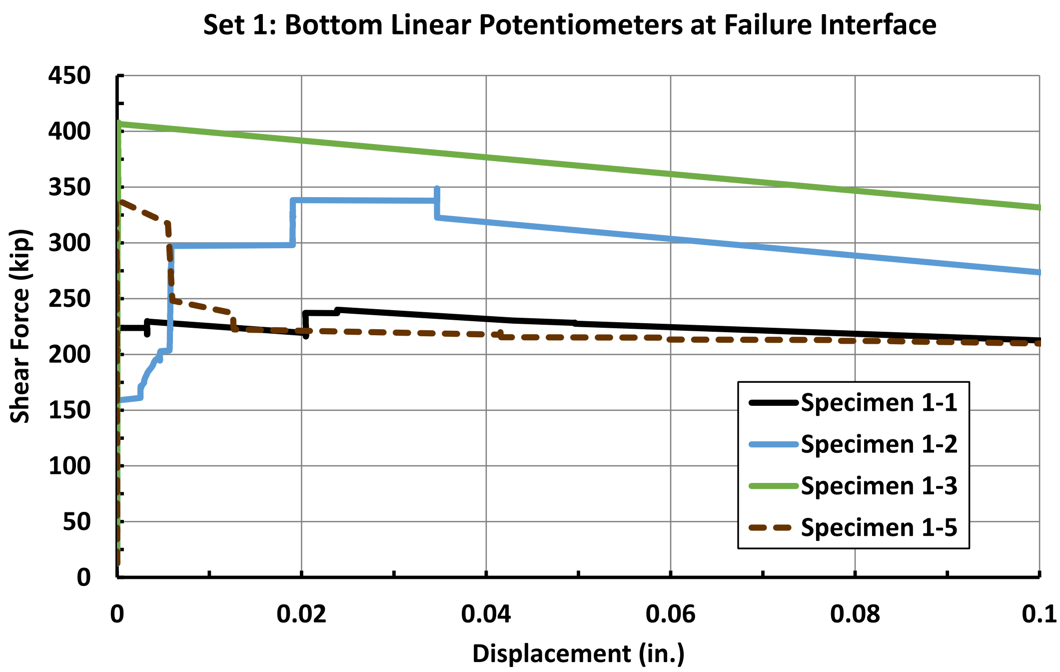


(a)

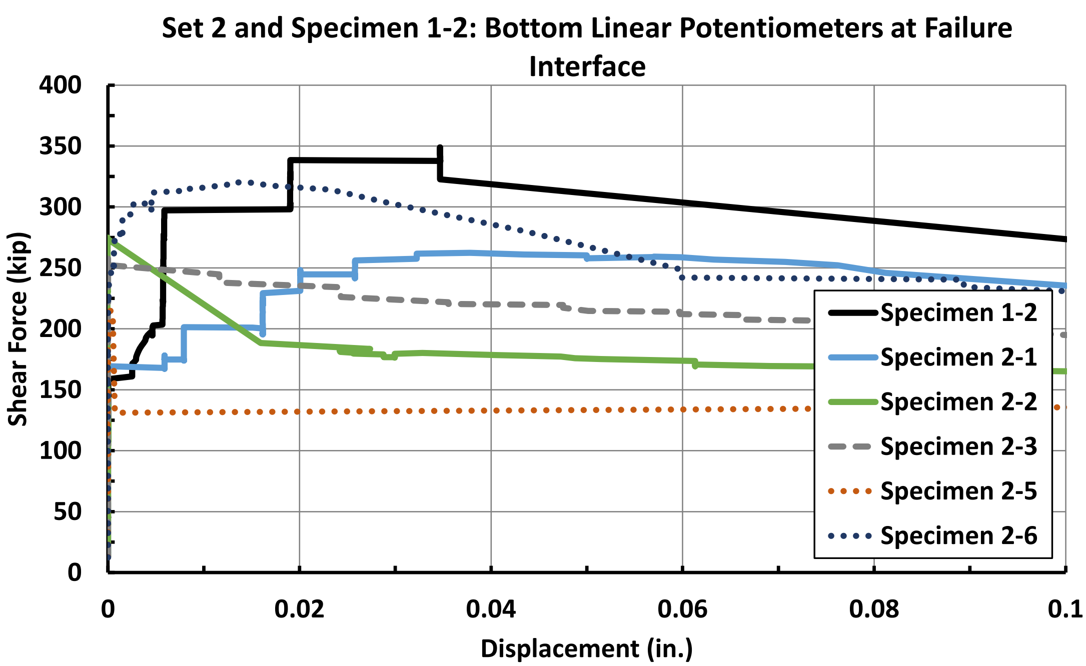


(b)

Figure F28: Top linear potentiometers measuring across the failure interface

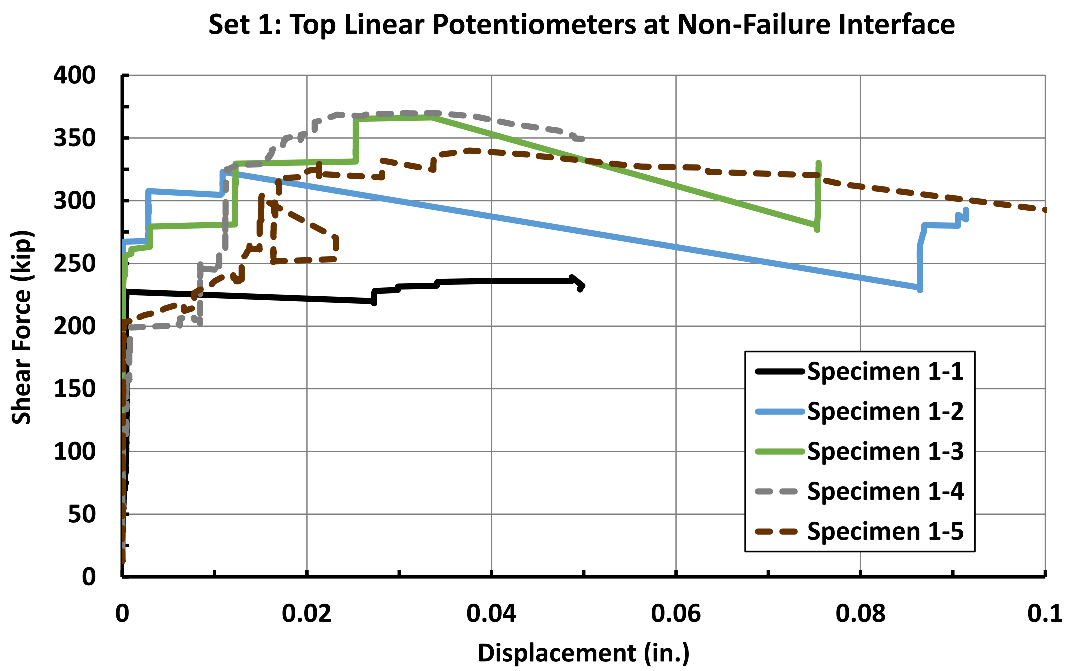


(a)

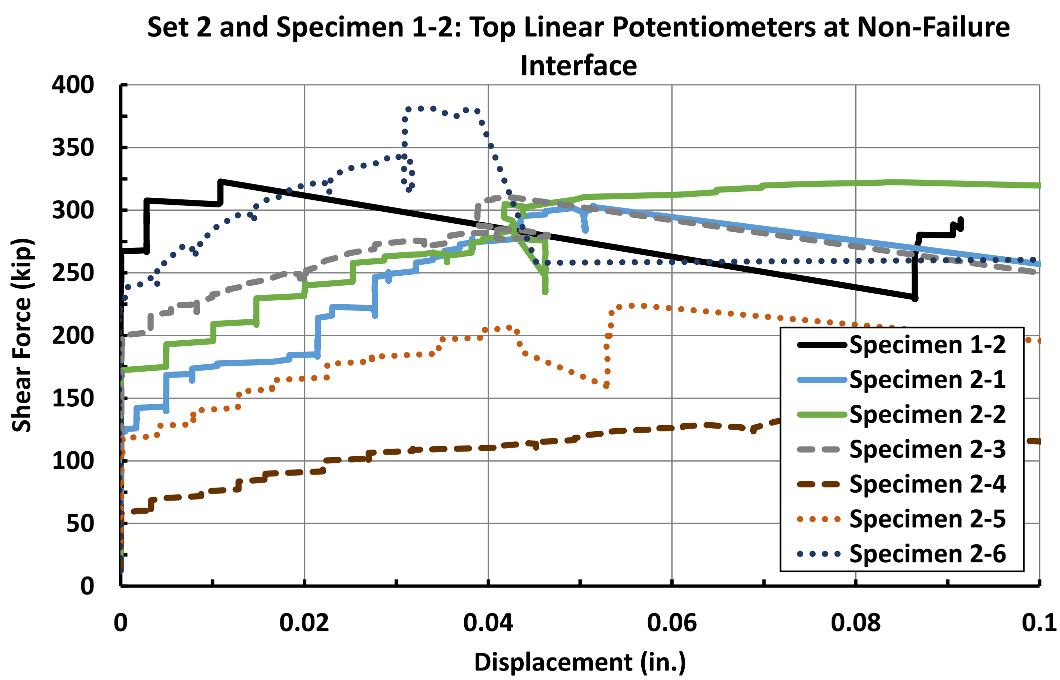


(b)

Figure F29: Bottom linear potentiometers measuring across the failure interface

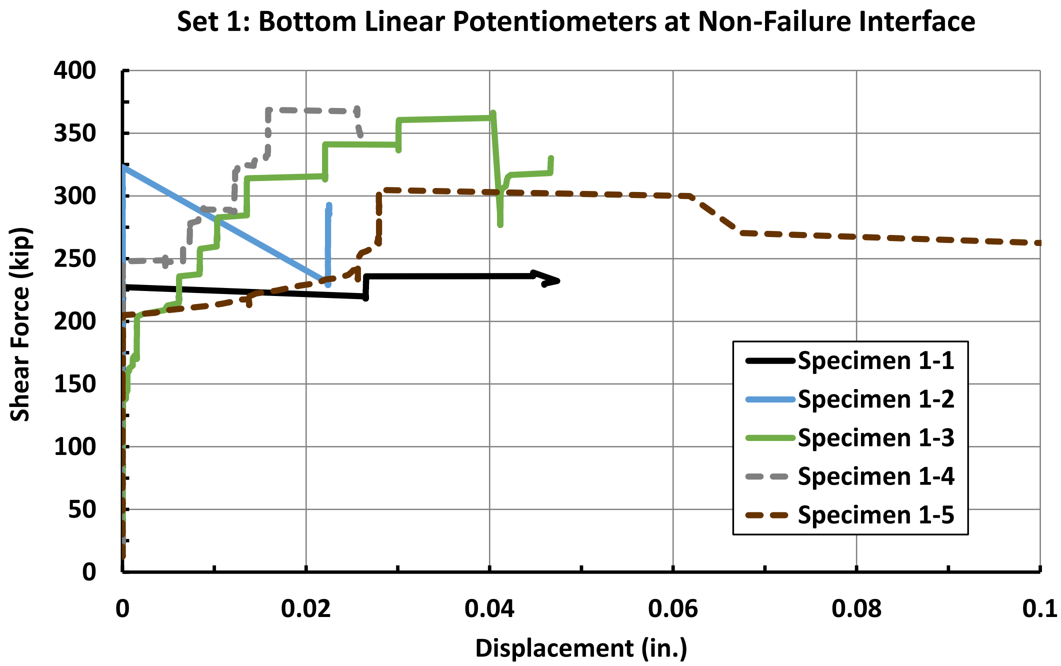


(a)

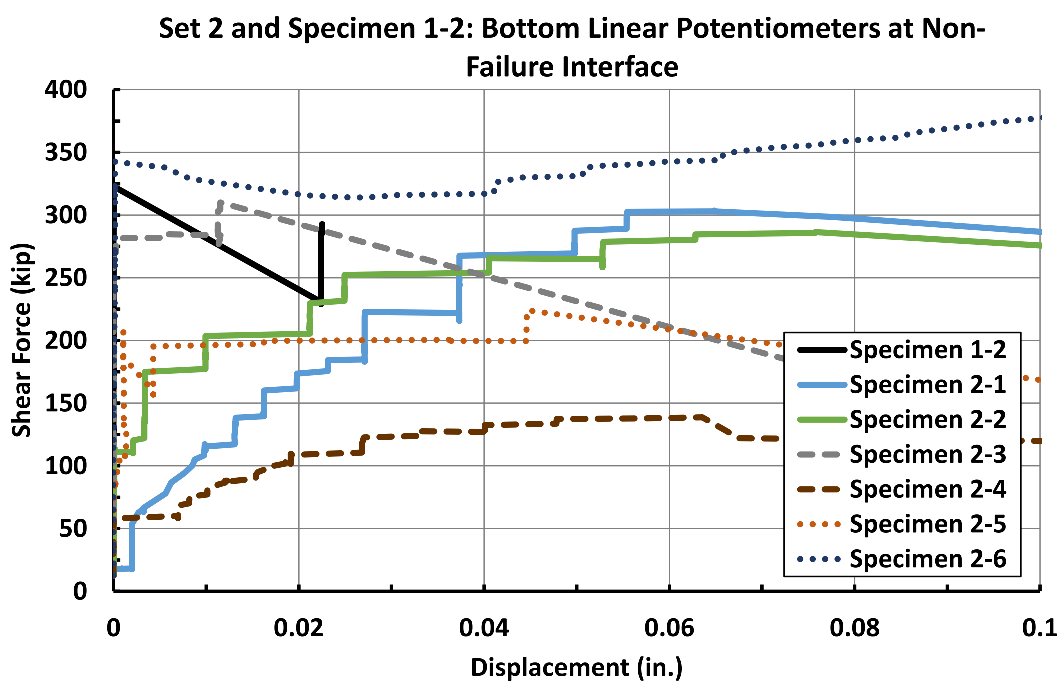


(b)

Figure F30: Top linear potentiometers measuring across the non-failure interface



(a)



(b)

Figure F31: Bottom linear potentiometers measuring across the non-failure interface

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